

**FIELD LOCATION & MARKING OF NO-PASSING ZONES DUE
TO VERTICAL ALIGNMENTS USING THE GLOBAL
POSITIONING SYSTEM**

A Thesis

by

CAMERON LEE WILLIAMS

Submitted to the Office of Graduate Studies of
Texas A&M University
in partial fulfillment of the requirements for the degree of

MASTER OF SCIENCE

May 2008

Major Subject: Civil Engineering

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Approved by:	
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ABSTRACT

Field Location & Marking of No-Passing Zones Due to Vertical Alignments Using the
Global Positioning System. (May 2008)

Cameron Lee Williams, B.S., Texas A&M University

Chair of Advisory Committee: Dr. H. Gene Hawkins, Jr.

Passing on two-lane roadways is one of the most difficult movements a driver may perform and guidance on where passing maneuvers are prohibited is given by the location of no-passing zones. Currently the processes for identifying no-passing zone locations can be daunting and many practices require work crews to operate in the roadway creating potentially hazardous situations. Due to these challenges new alternatives need to be developed for the safe, accurate, and efficient location of no-passing zones on two-lane roadways.

This thesis addresses the use of Global Positioning System (GPS) coordinates to evaluate sight distance along the vertical profile of roadways to provide an alternative for an automated no-passing zone location system. A system was developed that processes GPS coordinates and converts them into easting and northing values, smoothes inaccurate vertical elevation data, and evaluates roadway profiles for possible sight restrictions which indicate where no-passing zones should be located.

The developed automated no-passing zone program shows potential in that it identifies the general location of no-passing zones as compared to existing roadway markings.; however, as concluded by the researcher, further evaluation and refinement is needed before the program can be used effectively in the field for the safe, accurate, and efficient location of no-passing zones.

DEDICATION

To my Mom, Dad, and Brother

ACKNOWLEDGMENTS

There have been many people who have made this thesis possible and there is no way to remember them all; but I would like to take this opportunity to thank a few of them.

First I would like to thank my advisor, Dr. Gene Hawkins, Jr. You made transportation fun, interesting, and gave me the opportunity to work for you while pursuing my masters degree. It was a learning experience and I know I will use the lessons I have learned through out my career. Second I would like to thank my fellow graduate students. You made graduate school a memorable experience. Without your willingness to help and to listen, I am not sure this thesis and degree could have happened. The friendships formed will last a life time. Next, I would like to thank my church family at Cavitt church of Christ in Bryan. You gave me a home away from home during my years at A&M. You became a part of my family and gave me the perfect place to grow. For that I am grateful. Lastly, and most importantly, I want to thank my entire family, specifically my Mom, Dad, and Brother. My whole life you have always been there for me. You have disciplined me, you have encouraged me, and most importantly you have loved me. I would not be the person I am today without your support. Thank you for raising me in a Christian home and for providing me opportunities to refine my abilities, character, and faith. I pray that I will be able to use those gifts to honor your efforts and improve the quality of life of those who I may meet.

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CHAPTER I

INTRODUCTION

Passing on two-lane roadways is one of the most difficult movements a driver may perform because the vehicle must enter the opposing traffic stream to complete the maneuver. Solid centerlines on the roadway surface, also known as barrier lines, along with supplemental signing give guidance on where passing maneuvers are prohibited. These pavement markings and signs indicate to drivers where there is insufficient sight distance to complete a passing maneuver. The locations delineated by these markings are known as no-passing zones.

In 1999, two-lane roadways alone made up approximately 82 percent of the centerline miles in Texas (Fitzpatrick et al. 2001). No-passing zones must be located and accurately marked on the pavement surface of two-lane roadways to effectively assist drivers. However, locating no-passing zones can be a daunting and very time-consuming process under the current practices utilized by state transportation agencies. Additionally, many current practices require work crews to operate in the roadway creating potentially hazardous situations.

This thesis follows the style of the *ASCE Journal of Transportation Engineering*.

Due to these challenges, new alternatives need to be developed for the safe, accurate, and efficient location of no-passing zones on two-lane roadways. The use of Global Positioning System (GPS) coordinates to evaluate sight distance provides one such alternative for an automated no-passing zone location system and is the subject of this thesis.

Problem Statement

Many methods for locating no-passing zones are available and range in cost, time, and accuracy. Despite the many alternatives, new methods that will efficiently locate no-passing zones, define the no-passing zones accurately, and do so safely are needed.

GPS has the potential to meet these needs but processes for gathering roadway GPS data, smoothing GPS data, mathematically locating no-passing zones from GPS data, and field implementation of the results must be addressed. Theoretically, a system enabling work crews to drive two-lane roadways with GPS units to automatically determine no-passing zones can be developed by focusing on these issues.

Research Objectives

The goal of this research is to develop a system that utilizes GPS technology in locating no-passing zones associated with the vertical alignment of a roadway. Work crews should be able to safely, efficiently, and accurately locate no-passing zones on two-lane

roadways with the system while driving at highway speeds. Several objectives have been identified to achieve this goal.

They are:

- identify the processes necessary to smooth GPS data and geometrically model roadway surfaces,
- create an algorithm for locating no-passing zones from modeled roadway surfaces due to vertical sight obstructions, and
- validate the system by comparing results to existing no-passing zone pavement markings.

Thesis Organization

This thesis is divided into seven different chapters. Chapter I provides an introduction to the thesis and includes the problem statement, research objectives, and thesis organization. Chapter II provides background information and discusses the history of no-passing zones, no-passing zone location methods, GPS technology, and geometric modeling of roadways. Chapter III describes the processes used to collect and format GPS data. Chapter IV covers the development of the no-passing zone model including the necessary steps for smoothing GPS data. Also, the theory behind the algorithm created for locating no-passing zones from GPS data is explored in detail. Chapter V reviews the study design used for testing the developed no-passing zone location system and includes the testing sites, the GPS data collection equipment, and the analyses methodologies. Chapter VI provides analysis of the developed no-passing zone system

and covers the sensitivity and comparison tests that were completed. Lastly, chapter VII summarizes the research and provides some concluding thoughts along with recommendations for further research.

CHAPTER II

BACKGROUND

No-passing zones are not a new topic. There are many subtopics related to them which make the breadth of information pertaining to them quite large. The intent of this chapter is to pin down and discuss basic topics associated with this particular research in the development of an automated GPS no-passing zone location system.

When discussing no-passing zone locations, some confusion exists because there seem to be conflicting approaches between the American Association of State Highway Transportation Officials' (AASHTO) passing sight distances (PSD) and the Manual on Uniform Traffic Control Devices' (MUTCD) minimum no-passing zone distances. The criteria associated with these two guidelines are discussed. After covering the methodologies and theories behind the AASHTO and MUTCD guidelines, many of the more utilized no-passing zone location methods are explained. Also, the basics of how GPS technology works are outlined along with some of the issues associated with its use. Additionally, some discussion concerning completed research in the area of geometric roadway modeling using GPS and automated location of no-passing zones using GPS is provided. These topics should provide a more focused view of no-passing zones as they relate to the research at hand.

AASHTO: Passing Sight Distance in Design

Passing maneuvers are dependent on PSD which is a driver's ability to see the roadway ahead. Specifically, AASHTO in their book *A Policy on Geometric Design of Highways and Streets* (also known as the Green Book) defines PSD as follows:

“The passing driver should be able to see a sufficient distance ahead, clear of traffic, to complete the passing maneuver without cutting off the passed vehicle before meeting an opposing vehicle that appears during the maneuver (AASHTO 2004).”

No-passing zones occur when available PSD is less than a required minimum PSD.

When this sight deficiency arises, a passing maneuver may be hazardous to attempt.

AASHTO (2004) outlines procedures for implementing safe PSD in design. Listed below are some of the assumptions that are made concerning driver behaviors during passing maneuvers.

- the overtaken vehicle travels at uniform speed,
- the passing vehicle has reduced speed and trails the overtaken vehicle as it enters a passing section,
- when the passing section is reached, the passing driver needs a short period of time to perceive the clear passing section and to react to start his or her maneuver,

- passing is accomplished under what may be termed a delayed start and a hurried return in the face of opposing traffic. The passing vehicle accelerates during the maneuver, and its average speed during the occupancy of the left lane is 10 mph higher than that of the overtaken vehicle, and
- when the passing vehicle returns to its lane, there is suitable clearance length between it and an oncoming vehicle in the other lane.

Furthermore, AASHTO goes on to state that the minimum PSD is the combination of the following four distances:

- d_1 = distance traversed during perception and reaction time and during the initial acceleration to the point of encroachment on the left lane,
- d_2 = distance traveled while the passing vehicle occupies the left lane,
- d_3 = distance between the passing vehicle at the end of its maneuver and the opposing vehicle, and
- d_4 = distance traversed by an opposing vehicle for two-thirds of the time the passing vehicle occupies the left lane, or $2/3$ of d_2 .

The minimum PSDs that result from these assumptions are seen in Table II-1.

Table II-1. AASHTO Minimum Passing Sight Distances

Design Speed (mph)	Passing Sight Distance (ft)
20	710
25	900
30	1090
35	1280
40	1470
45	1625
50	1835
55	1985
60	2135
65	2285
70	2480
75	2580
80	2680

*Taken from Exhibit 3-7 of AASHTO Green Book

AASHTO's assumptions lead to what many consider to be very conservative PSD requirements. These distances differ from the guidelines in the MUTCD for marking no-passing zones.

MUTCD: No-Passing Zone Marking Criteria

The 2003 MUTCD does not provide information concerning how the minimum PSD guidelines presented in Part 3 were developed (Hardwood et al. 2005). Hardwood et al. mentions that numbers in the current MUTCD are identical to the 1940 American Association of State Highway Officials (AASHO) policy. Hardwood et al. conclude that the minimum sight guidelines presented by the 1940 AASHO policy are a compromise between the distances required for a flying pass and those of a delayed pass. A flying pass is a case where a passing driver approaches the vehicle to be overtaken and

proceeds to the left lane and around the overtaken vehicle without having to slow down.

A delayed pass is one in which the passing vehicle approaches the vehicle to be overtaken, decelerates to follow the vehicle to be overtaken, and then accelerates to complete a pass.

The *Texas Manual on Uniform Traffic Control Devices* (2006) which, except for the speed of 25 mph, has exactly the same minimum PSDs as the 2003 MUTCD states the following in reference to the accepted minimum PSD as seen in Table II-2:

“Distances shown are the minimum warrants based on AASHTO’s decision sight distances for a rural road avoidance maneuver (speed/path/direction change). The distances are derived for traffic operation-control needs and should not be confused with design passing sight distances which are based on different assumptions.”

This statement helps provide some clarification for the differences between AASHTO’s PSD design guidelines and the MUTCD’s standards for minimum PSD.

Table II-2. Texas MUTCD Minimum Passing Sight Distances

Design Speed (mph)	Passing Sight Distance (ft)
25	450
30	500
35	550
40	600
45	700
50	800
55	900
60	1000
65	1100
70	1200

* Values from the 2006 Texas *MUTCD*

No-Passing Zone Location Methods

As for the actual implementation of locating no-passing zones there are many possible methods. A few of the available options include the walking method, the eyeball method, the speed and distance method, single vehicle method, and multi-vehicle method (Traffic Control Devices Handbook 2001).

Walking Method

The walking method requires two crewmen to walk the centerline of a roadway with a chain or rope stretched between workers. Each crewmember holds a target that is 3.5 feet in height which represents a driver's eye height for the rear crewman and an oncoming vehicle for the lead crewman. A no-passing zone is located when the rear crewman can no longer see the lead crewman's target due to horizontal or vertical sight obstructions. A no-passing zone ends when the lead crewman's target comes back into

view of the rear crewman. This process is very time consuming and can be hazardous since crew members must work in the roadway.

Eyeball and Speed and Distance Methods

The eyeball method requires the roadway be driven in a vehicle. While driving, the person(s) in the vehicle locates visually, by judgment, where no-passing zones begin and end. This method is quick and inexpensive, but is very dependent on the experience of the person(s) completing the evaluation (Traffic Control Devices Handbook 2001).

Similarly, like the eyeball method, the speed and distance method requires an individual to initially estimate the beginning location of a no-passing zone. While stationary, a person records the speed of a receding vehicle and the amount of time the receding vehicle takes to travel from the estimated beginning of the no-passing zone to the point where the vehicle disappears from view. The recorded speed (S) in mph and time (t) in seconds are plugged into Equation II-1 to get distance (D) in feet. Through an iterative process and based on the minimum PSD as set by the MUTCD, the beginning of a no-passing zone can be determined in relation to the disappearing location of the receding vehicle (Brown & Hummer 2000).

$$D = 1.47 \times S \times t \quad (\text{II-1})$$

Single Vehicle Method

The single vehicle method calls for vehicles to be outfitted with Distance Measuring Instruments (DMI). Using these vehicles with DMIs, operators can drive through a curve (vertical or horizontal) until the view opens up and allows sufficient sight distance for passing; this is the end of a no-passing zone. From this location, a person can use the DMI to measure a distance equal to the minimum PSD and then mark the beginning of a no-passing zone in the opposing direction. This process is repeated for both travel directions of a roadway to locate no-passing zones. Like previous methods, this method is dependent on the judgment of the person operating the vehicle.

Multi-vehicle Method

There are several multi-vehicle no-passing zone location methods available. One of these methods utilizes two vehicles with DMIs. While communicating via two-way radio, the two vehicles traverse a roadway while maintaining, according to the DMIs, spacing equal to the required minimum PSD. Based upon when the lead vehicle disappears and reappears from the lagging vehicle's view; the beginning and end of no-passing zones can be established accordingly (Brown & Hummer 2000).

There is no one method that is overwhelmingly used by state transportation agencies. Results from an informal survey conducted for a Texas Transportation Institute (TTI) report show that agencies were using single or multi-vehicle methods that utilize some combination of distance measuring instruments and human sight to locate no-passing

zones (Rose et al. 2004). These methods do get work crews out of the roadway, as compared to the traditional walking methods, but still rely on judgment to determine the beginning and ending of no-passing zones.

Global Positioning System (GPS)

GPS is a technology that allows for any position on the earth to be determined.

Consisting of three segments that are defined as space, control, and user, GPS was initially developed by the Department of Defense for military purposes in the 1970s (El-Rabbony 2006).

GPS Operations

The space segment consists of 24 satellites circling the earth in six different orbital planes while transmitting a microwave radio signal that at the very least contains two carrier waves, two digital codes, and a navigation message. The digital codes and navigation message are attached to the carrier waves. The main purpose of the digital codes is to allow for calculation of distances between receiver and satellite. The navigation message contains information as a function of time. Meanwhile, the control segment consists of ground bases that track satellite positions and are spread across the world allowing the GPS satellites to be tracked at all times. Additionally, the ground stations send corrections and instructions to the satellites. The user segment is any receiver of the GPS satellite signals and can, through onboard software, calculate positional data based on the information obtained from the GPS signal. In order to

achieve positional data, signals from at least three satellites must be obtained so that triangulation can be used to locate a receiver's position. Furthermore, a receiver must be processing signals from at least four satellites to obtain elevation data from GPS (El-Rabbany 2006).

When using GPS there are possible errors involved. These errors can be grouped into three categories: satellite, receiver, and atmospheric. Satellite errors are attributed to the clocks in the satellites and ephemeris errors. GPS satellites have atomic clocks on board. Since the types and combinations of clocks have changed slightly with each new GPS satellite developed, it largely depends on the model of the satellite. However, because the clocks are not perfect, this introduces error into coordinate values since GPS calculations are dependent on time. Ephemeris errors refer to errors in GPS satellite errors. Precisely predicting the orbits of satellites is impossible due to the changing forces put upon them by the elements of space; however, as the number of ground monitoring stations increase, estimates predicting satellite orbits will become ever better (El-Rabbany 2006).

Errors that fall into the category of those introduced by the GPS receiver are receiver clock error, multipath error, and antenna phase center error. GPS receiver clocks are not of the same quality as those in GPS satellites. Therefore, their inability to correctly report time also introduces error into GPS calculations. Multipath error refers to the line of path that GPS signals take to receivers. Depending on the location, the signals may

bounce off of buildings or water surfaces before arriving at GPS receivers. If this occurs, then error is once again introduced because true time delays between satellite and receiver are not reported. Antenna phase center errors are concerned with the fact that the location on the GPS antenna where the GPS signal is actually received is not always at the geometric center of the antenna. This will change with orientation of the satellite to the antenna (El-Rabbany 2006), but this is usually a small error.

As mentioned previously, the atmospheric conditions also insert error into GPS positional data and are referred to as ionospheric and tropospheric delays. The ionosphere is an electron dense part of the atmosphere that is present from about 31 miles to approximately 621 miles above the earth's surface. This region causes the GPS satellite signal to bend, speed up, and slow down resulting in distortions of the ranges between receiver and satellite. The troposphere is the region from the surface of the earth to about 31 miles above the surface. Temperature, pressure, and humidity in this region act to slow down the GPS signal which, as with the ionospheric delay, distorts the ranges between receiver and satellite (El-Rabbany 2006).

Lastly, there is an additional error which must be mentioned that does not quite fall into the above three categories. It may not really be classified as an error but affects the accuracy of GPS positional data. The geometry of the satellites' orientation with respect to the receiver will affect the quality of the calculated information. The measure of this is referred to as the dilution of precision which can be further broken down into the two

categories of horizontal dilution of precision and vertical dilution of precision. The basic idea is that the more spread out the geometry of the satellites are with respect to the GPS receiver, the better the positional data will be (El-Rabbany 2006).

Previous Research

To determine where no-passing zones are located from GPS data, a model of a roadway's geometry must be created. After modeling a roadway, locations that have PSD restrictions can be determined allowing for the establishment of no-passing zones.

Ben-Arieh et al. (2004) developed a geometric model of roadways that utilized b-spline curves which are a piecewise polynomial function. The model requires that GPS data first be cleaned so that control points can be established and used by b-spline curves to model a roadway's geometry. Two methods were used to clean the GPS data. One method examines the elevation differences between successive points. If the difference is greater than 16.4 feet, the GPS data point in question is considered an outlier. The second method utilizes quadratic regression to clean the data and two sigma residuals are used as cut off limits for determining whether a GPS point is an outlier (Ben-Arieh et al. 2004). After cleaning the data, control points are established based on equal spacing and are used by the b-spline curves to model a roadway's geometry.

Upon creation of the roadway model, two methods were used for validation. One of the validation practices is simply accomplished through visual inspection while the other

method compares the model vertical profile to design plan profile (Young 2004). In comparing the model vertical profile to the design plan profile, two different errors are present and are called bias and variance. Bias refers to the average difference between model profile and design profile elevations and in the study ranged from -0.52 feet to 17 feet on the three evaluated roadways. Variance could be better referred to as the adjusted error. After subtracting out the bias, the variance was the average difference at each point along the roadway and ranged from 1.64 ft to 3.94 ft for the three roadways. It was concluded by Chang (2004) that the adjusted error and variance were so small that they could be neglected.

Nehate and Rys (2006) used the geometric model developed by Ben-Arieh et al. to calculate stopping sight distances (SSD) on roadways. Later, Namala and Rys (2006) took the stopping sight distance model and expanded upon it to include PSD. Once sight distances were calculated, no-passing zones could be determined for the roadway. Namala and Rys included a minimum SSD in their PSD model which is not usually done. The model was completed according to AASHTO design guidelines for PSD and the MUTCD for marking no-passing zones. In practice, the MUTCD is the only guide used for marking no-passing zones in the field. Conclusions were made that the no-passing zones found by the developed models were in “general agreement” with existing conditions and the differences were attributed to the conservativeness of including SSD and the fact that the GPS data from KDOT data logs were aligned with the data collected on existing conditions. In further examination of reported results for one of the

roadways in which model results were compared to existing conditions, the differences in no-passing zone lengths varied from approximately 150 ft to 1600 ft. If the no-passing zones found by the model at locations where existing conditions do not currently show no-passing zones are considered, the differences go as high as 2400 feet (Namala & Rys 2006). These results do not seem reasonable for the actual field marking of no-passing zones. Kansas Department of Transportation (KDOT) use of this research has been to rate roads for consideration of improvements based on sight deficiencies found from the models. The developed models have not been used in the field for the marking of no-passing zones.

At Kansas State University (KSU) further research was completed that looked into the development of a different three-dimensional roadway model. This research addressed many of the errors associated with GPS data and allowed for the development of methods to combine historically logged roadway data from KDOT (Young 2004). According to Young (2004), the geometric model developed for the KDOT SSD and no-passing zone applications was not adequate for addressing errors in the GPS data that are referred to as parallel traces, offset traces, and discontinuities. Also, according to Young, the original KSU model performed better on single GPS data runs than when multiple GPS data runs were combined. In response, Young came up with a model that had the ability to utilize the historic roadway inventory GPS data that has been collected through KDOT's pavement condition survey and video log survey.

Young used a combination of techniques that enable the use of the historic GPS data to create a three-dimensional geometric roadway model. These techniques addressed the errors of parallel traces, offset traces, and discontinuities. Young built the model on the premise that GPS data collected within a small time frame is highly correlated and that alternatively, GPS data collected at sufficiently different times have independent errors. For example, if multiple GPS runs of the same roadway are collected within 15 minutes of each other, or even an hour, they should have similar errors and have relative accuracy when compared to one another. However, if the data are compared for absolute accuracy the results would not be the same. As such, if multiple runs from different days or different years are combined, then more absolute accurate data can be obtained; as the number of samples containing independent errors increase, the absolute accuracy will increase.

The draw back is that this does not benefit the current goal of developing a method which allows no-passing zones to be marked using GPS data collected in the field. Sufficiently long time periods between GPS data runs on roadways to attain more accurate roadway models would be counterproductive. This leaves room for current research to develop methods and models that meet the needs of using GPS to mark no-passing zones in the field within short time frames.

There has been other work completed in the area of modeling roadway geometrics. Easa et al. (1998) developed a two-step analytical model for creating vertical profiles from

field data. First, the model divides the roadway into segments of tangents, crest curves, and sag curves based on trends in incremental slopes of field data points. Once roadway segments are identified, the segments are fit according to whether they are tangents or curves. Linear regression is used to fit tangents while splines are used to model the vertical curves. One problem in using this method is that raw GPS data collected from a moving vehicle is not smooth. As a result, the determination of trends for tangents, crest curves, and sag curves is very difficult and not believed by the researcher to be feasible with the current positional data produced by GPS systems.

Easa (1999) did additional work in identifying the vertical profiles of roadways from field data. Easa developed algorithms for determining the optimum vertical alignment from field data based on optimization methods. Easa used the knowledge that vertical profiles consist of linear tangent sections and parabolic curves. Using the roadway design equations for these two mathematical representations, Easa created a linear optimization method that found the optimum values for incoming and outgoing slopes as well as the optimum elevations of the given vertical points of curvature and tangency. Easa did suggest that the problem could be taken a step further by creating a non-linear optimization problem by varying the station location of the vertical points of curvature and tangency instead of fixing them in place. This process can be very time consuming since it is an iterative process and on a roadway with multiple vertical curves the process would become even more difficult.

CHAPTER III

DATA COLLECTION AND FORMATTING

This chapter provides specific information on the GPS equipment used for data collection, the data conversion process, and the methods of calculating station values for vertical roadway profiles. In discussing the data conversion process, the equations and variables used are given along with an explanation for choosing the specified projection. The information provided is essential in understanding what data format is needed for the no-passing zone program and gives insight into the required steps for collecting and prepping the data.

GPS Equipment

GPS has continually been developed and improved since the 1970s. As such, there are many different GPS technologies available that range in capabilities and costs.

Countless hours and money could be placed into evaluating, comparing, and analyzing various GPS receivers. The focus of this research however, is the development of an automated no-passing zone location model. So, even though the selection of an effective GPS unit is extremely important to the research, GPS options were not exhaustively pursued. Instead, a GPS unit which was readily available and met the researcher's needs was utilized.

The Center for Transportation Safety at the TTI has an instrumented vehicle equipped with a Trimble DSM 232 Differential Global Positioning System (DGPS), as seen in

Figure III-1, which uses commercial satellite differential correction services provided by OmniSTAR.



Figure III-1. Instrumented Vehicle

This GPS system uses OmniSTAR's lowest grade correction service, Virtual Base Station (VBS) differential correction. OmniSTAR VBS differential correction service is a commercial measurement domain wide area differential GPS (El-Rabbany 2006). OmniSTAR maintains a base station network across the country which receives GPS signals. Since the base station locations are known, corrections can be calculated to improve the accuracy of received GPS signals. These corrections are beamed to OmniSTAR satellites which broadcast corrections to individual GPS units, such as the Trimble 232 DMS unit. This technology allows for increased GPS accuracy of collected

data. According to the DSM 232 specifications sheet, with VBS correction the horizontal accuracy is typically better than three feet. Additionally, the unit collection rate capability is ten hertz, which means the unit can record ten separate data points every second.

Even though the GPS receiver and antenna are really the only necessary equipment for the research, TTI's instrumented vehicle has several other tools that are beneficial to the research. In addition to the GPS the vehicle is equipped with a Dewetron computer, DMI, and video collection capabilities. These tools allow for not only the collection of GPS data, but also video data of the roadway with DMI readings superimposed on the screen. The Dewetron, DMI, and GPS receiver can be seen in Figure III-2.

GPS Data Conversion

GPS output provides horizontal positional data in latitude and longitude values while vertical data are reported in meters above sea level. This form of positional data is not useable for inclusion in a no-passing zone location program. Therefore the GPS positional output data is converted into northing and easting values, and to do this a projection was selected. Since the no-passing zone location program is being developed for use in Texas, the Texas Centric Mapping System/Lambert Conformal (TCMS/LC) was chosen as the final coordinate system. TCMS/LC can be used for the entire state and does not limit the use of the program to one geographic region in the state. The equations for converting the GPS longitude and latitude values into northing and easting

values are seen below in Equations III-1 through III-15. Additionally, the constants that are unique for the TCMS/LC and needed for the conversion process are located in Table III-1.

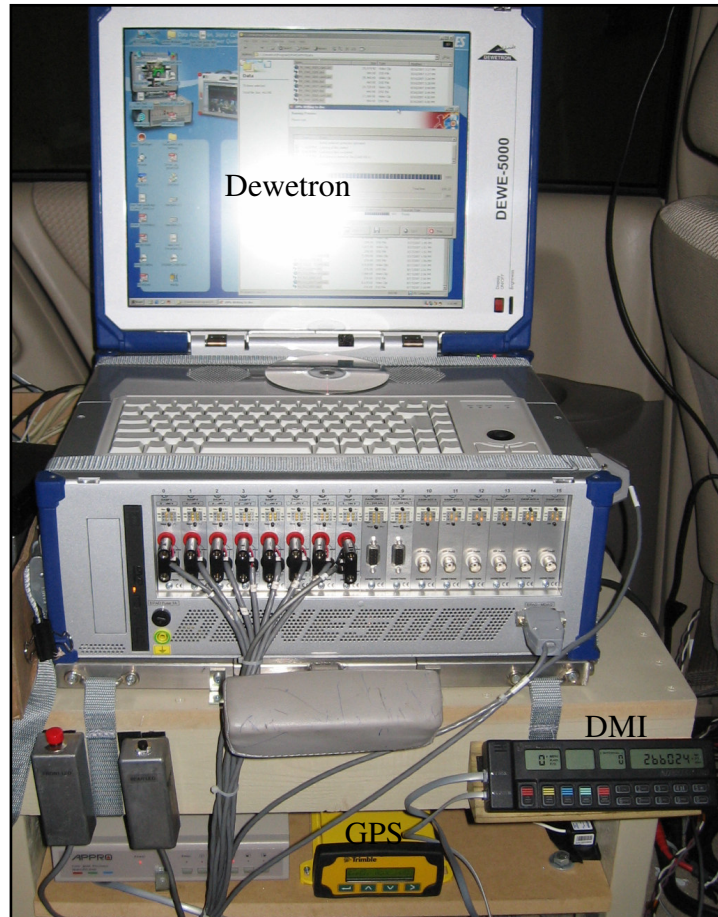


Figure III-2. Dewetron Computer, GPS, and DMI

Table III-1. Texas Centric Mapping System-Lambert Conformal Variables

Variable	Symbol	Value
First eccentricity	e	0.081819191
Flattening	f	1/298.257222101
Latitude Grid Origin	ϕ_0	18° N
Longitude Grid Origin	λ_0	100° E
Northern Latitude Parallel	ϕ_N	35° N
Southern Latitude Parallel	ϕ_S	27.5° N
False Easting	E_0	1500000 m
False Northing	N_b	5000000 m
Latitude of Point	ϕ_P	Varies
Longitude of Point	λ_P	Varies

*Datum: NAD83(WGS-84) Ellipsoid: GRS80 (Wolf & Ghilani 2006)

In order to complete these conversions for processing two custom Excel functions were written in the Microsoft Visual Basic Editor. The two functions are as follows:

- Easting(latdeg,longdeg)
- Northing(latdeg,longdeg)

These functions utilize the latitude and longitude of each collected GPS point in decimal degrees and give their corresponding easting and northing values in feet according to the values in Table III-2 and the conversion equations presented.

$$w_1 = W(\phi_S) = \sqrt{1 - e^2 \sin^2(\phi_S)} \quad (\text{III-1})$$

$$w_2 = W(\phi_N) = \sqrt{1 - e^2 \sin^2(\phi_N)} \quad (\text{III-2})$$

$$m_1 = M(\phi_S) = \frac{\cos(\phi_S)}{W(\phi_S)} \quad (\text{III-3})$$

$$m_2 = M(\phi_N) = \frac{\cos(\phi_N)}{W(\phi_N)} \quad (\text{III-4})$$

$$t_o = T(\phi_0) = \sqrt{\left(\frac{1 - \sin(\phi_0)}{1 + \sin(\phi_0)}\right) \left(\frac{1 + e \sin(\phi_0)}{1 - e \sin(\phi_0)}\right)^e} \quad (\text{III-5})$$

$$t_1 = T(\phi_S) = \sqrt{\left(\frac{1 - \sin(\phi_S)}{1 + \sin(\phi_S)}\right) \left(\frac{1 + e \sin(\phi_S)}{1 - e \sin(\phi_S)}\right)^e} \quad (\text{III-6})$$

$$t_2 = T(\phi_N) = \sqrt{\left(\frac{1 - \sin(\phi_N)}{1 + \sin(\phi_N)}\right) \left(\frac{1 + e \sin(\phi_N)}{1 - e \sin(\phi_N)}\right)^e} \quad (\text{III-7})$$

$$n = \frac{\ln(m_1) - \ln(m_2)}{\ln(t_1) - \ln(t_2)} \quad (\text{III-8})$$

$$F = \frac{m_1}{n \times t_1^n} \quad (\text{III-9})$$

$$R_b = aFt_0^n \quad (\text{III-10})$$

$$\gamma = (\lambda_0 - \lambda_p)n \quad (\text{III-11})$$

$$t = T(\phi_P) = \sqrt{\left(\frac{1 - \sin(\phi_P)}{1 + \sin(\phi_P)}\right) \left(\frac{1 + e \sin(\phi_P)}{1 - e \sin(\phi_P)}\right)^e} \quad (\text{III-12})$$

$$R = aFt^n \quad (\text{III-13})$$

$$E_P = R \sin(\gamma) + E_0 \quad (\text{III-14})$$

$$N_P = R_b - R \cos(\gamma) + N_b \quad (\text{III-15})$$

The TCMS/LC can be reported in either metric or US customary units. US customary units were selected as the desired output.

Station Calculation

The current no-passing zone model only focuses on the PSD restrictions caused by the vertical profile of a roadway. The inputs required by the model are stations and elevations. In typical preparation of plan and profile sheets, stationing is based on distances in the horizontal plane. This method is used in the model. The collected GPS data provides three-dimensional points successively along the roadway; however, since stationing is only calculated in the horizontal plane, the northing and easting coordinates are the only values needed. To determine stationing incremental distances are calculated between successively collected GPS data points and are done so via a simple distance formula as is seen in Equation III-16 below.

$$\Delta_i = \sqrt{(x_{i+1} - x_i)^2 + (y_{i+1} - y_i)^2} \quad (\text{III-16})$$

In Equation III-16, x corresponds to easting values in feet and y corresponds to northing values in feet. After calculating the incremental distances between data points as they were successively collected, the incremental values are cumulatively added together to determine station values at each data point in reference to the first data point. The result is that each x,y,z data point have an associated station. These station and elevation data points can be input into the no-passing zone program.

In order to speed the incremental distance calculations along a custom function was again written in Excel Microsoft Visual Basic Editor. It is called DeltaDist and requires

the input of x_1 , x_2 , y_1 , and y_2 where these values correspond to the easting and northing values of successive GPS points. The resulting output is the incremental distance between the two successive points.

CHAPTER IV

NO-PASSING ZONE MODEL DEVELOPMENT

The no-passing zone program consists of several steps that can be broken down into two general groups. These two groups include the creation of a smooth vertical profile and the no-passing zone algorithm. Smooth data is needed before the no-passing zone algorithm can be utilized effectively. Presented in this chapter are the processes that were used to create the needed smooth profile. Also, the methodology and theory of how the no-passing zone program works is delivered. This chapter should provide understanding to what takes place in the no-passing zone program.

Smoothing GPS Data

GPS data collected from a moving vehicle are not smooth and as covered earlier, GPS has errors inherent to the system. These errors can be minimized through various methods, but many of these methods utilize local base stations set up on a project's site. Such methods allow comparisons between true accurate data and collected data at a base station which enable corrections to be applied to data collected from a roving vehicle. However, due to the nature of this particular research application setting up a base station at each site is not reasonable. The current proposed solution is to use the OminSTAR satellite correction service to help offset some of the errors. Despite the use of a satellite correction service the GPS data is still rough. Because of these problems, the GPS data must be smoothed to produce a usable vertical profile. Roadway profiles, especially long segments, are unique in that they consist of multiple combinations of

tangents and parabolic curves. These multiple segments do not lend themselves to being modeled or smoothed by a single function. As a result, local regression methods have been selected for the smoothing of GPS data. More specifically, robust Loess regression is used.

Loess regression is a locally weighted nonparametric regression method where a span, also referred to as bandwidth, of points on either side of a test point is weighted. Data points closer to the test point are more heavily weighted than points near the edge of the bandwidth. A new value for the test point is calculated using the surrounding weighted points and regression analysis. Typically, either linear or quadratic least-squares regressions are used in Loess regression. Least-squares regression is a method for finding a unique optimum known function fit, or solution, to collected data points. This is achieved by minimizing the sum of the squared residuals of collected data points. Residuals refer to the differences between collected and fitted data values (Chapra and Canale 2002).

For this smoothing process, quadratic fitting was selected because parabolic curves are used in geometric design of vertical roadway profiles. If linear least-squares were used it would be impossible to fit and model vertical curves that occur in a roadway's profile and the capability of the smoothing process would be inadequate for this application. A quadratic fit has the capacity to fit either a straight line tangent or a parabolic curve.

This characteristic of quadratic least-squares makes it more suitable for use in smoothing roadway vertical profiles since they contain both tangents and curves.

Since Loess is a localized method, segments of a roadway are iteratively evaluated one at a time. This means that in smoothing a single data point, the smoothing process will only evaluate a snapshot of the roadway's profile. In doing this a roadway's vertical profile can remain intact because it is not possible, at least in this approach, to evaluate the entire profile at one time. Due to this approach the bandwidth becomes very important because the bandwidth distance defines what length of roadway should be considered in smoothing a single data point (Cleveland 1993).

Selecting the bandwidth could be an endless and frustrating task and early on in the creation of the smoothing process the researcher attempted to evaluate the effects of various combinations of smoothing iterations and bandwidth lengths; however, it was very difficult to identify any clear trends in the selection of an appropriate bandwidth. In response it was concluded that a bandwidth was needed which reflected typical lengths of design curves. When looking at design methods for vertical curves, the curve length selected is based on the design speed stopping sight distance and the algebraic differences between incoming and outgoing tangent slopes. In addition to these factors, there could be many other considerations such as right-of-way or budget restrictions which might influence design decisions. In reviewing some vertical profile design

sheets it was felt that most high-speed two-lane roads will have vertical curve algebraic differences that range from two to six.

Using design equations from the Green Book and the above mentioned algebraic differences, calculated curve lengths ranged from 320 feet to 2000 feet depending on the design speed of the roadway. It is important to remember these are minimum values and do not include outside factors which may effect final design selections. As can be seen, this range of curve lengths is quite large; however, despite the large range the decision was still made to attempt selection of a bandwidth that works for most vertical curve scenarios.

Based on the difficult encounter in identifying a proper bandwidth length, the researcher selected 900 feet for several reasons. First, a precedent was set in research conducted at KSU. In the process of cleaning the data for their geometric model, KSU researchers applied a quadratic regression and selected 984 feet because they stated research has found this distance approximately makes up an identifiable segment of roadway, such as a tangent or curve. Second, in collecting data at 60 mph, this bandwidth distance gives approximately five data points either side of the test point which was felt to be an adequate number for smoothing. Lastly, if the bandwidth is too small, then not enough data points will be used in the smoothing process and the effect of smoothing will be lost. This means if a vertical curve is present; the smoothing process may not recognize it and could represent it as a tangent. Alternatively, if the bandwidth is too large then the

smoothing process runs the risk of smoothing out the data too much and distorting the roadway geometry. Based on these reasons and judgment, the bandwidth of 900 feet seemed to provide a good compromise between these two extremes and a representation of the typical vertical curve lengths that might be encountered.

Below are the equations used in Loess regression smoothing. In Equation IV-1, x is the test point to be smoothed and x_i is a point within the bandwidth to be weighted. This equation determines the absolute value distance from x to x_i . Equation IV-2 finds the maximum value from the calculation of all x_i 's in Equation IV-1. This maximum difference in distance of the x_i 's from the test point and is used by Equation IV-3 to scale distances of each bandwidth point to the test point. Equation IV-4, known as the tricube weight function, uses Equation IV-3 values in calculating neighborhood weights. The bandwidth point farthest from the test point will have a weight of zero and those closest to the test point will have weights closer to one.

$$\Delta_i(x) = |x_i - x| \quad (\text{IV-1})$$

$$\Delta_q(x) = \max \text{ of } \Delta_i(x) \quad (\text{IV-2})$$

$$u_i(x) = \frac{\Delta_i(x)}{\Delta_q(x)} \quad (\text{IV-3})$$

$$w_i(x) = \begin{cases} (1 - |u_i(x)|^3)^3 & \text{for } |u| < 1 \\ 0 & \text{for } |u| \text{ otherwise} \end{cases} \quad (\text{IV-4})$$

After calculating neighborhood weights for each bandwidth point, Equation IV-5 determines the least squares quadratic fit. The new quadratic equation, Equation IV-6, predicts a new value at test point x . For this application, a new elevation value would be predicted for the station value at test point x .

$$\text{Min} \sum_i^n w_i(x) \times (y_i - a - bx_i - cx_i^2)^2 \quad (\text{IV-5})$$

$$\hat{y}(x) = a + bx + cx^2 \quad (\text{IV-6})$$

The above process may be influenced by outliers. To offset outlier affects in smoothing, a robust Loess regression method is applied which requires additional calculations.

After determining the fitted quadratic equation, Equation IV-7 calculates absolute value residuals at each point within the bandwidth, and Equation IV-8 determines the median residual value. The residuals, ϵ_i , and median residual, s , are plugged into Equation IV-9. Equation IV-9 values are inserted into Equation IV-10, known as the bisquare function, to calculate robust weights. A residual larger than $6s$ will have weight of zero and larger weights will be applied to smaller residuals.

$$\hat{\epsilon} = |y - y_i| \quad (\text{IV-7})$$

$$s = \text{median}(\hat{\epsilon}_i) \quad (\text{IV-8})$$

$$v_i = \frac{\hat{\epsilon}_i}{6s} \quad (\text{IV-9})$$

$$r_i(v) = \begin{cases} (1-v^2)^2 & \text{for } |v| < 1 \\ 0 & \text{for } |v| \text{ otherwise} \end{cases} \quad (\text{IV-10})$$

An iterative process follows calculation of the robust weights in which the Loess neighborhood weights are multiplied by the robust weights. This newly combined weight is used by least squares regression to predict a new quadratic fit for the data. The process of calculating residuals and new robust weights via bisquares is repeated and then combined with the original Loess neighborhood weights to calculate new least squares regression fit. This process is completed using MATLAB's "smooth" function where MATLAB completes five bisquare robust weight iterations.

Figure IV-1 provides an example of how the smoothing process works. Depicted is a vertical curve with unsmooth data. Two smoothing iterations are shown and demonstrate how the smoothing process works to calculate new elevation data without changing station values.

After smoothing the GPS data by two robust Loess regression smoothing processes, Loess regression is used to generate new estimated elevation points at evenly spaced station values. The smoothing process only affects the elevation values of the collected data; it does not affect station values. The distance between collected data points is a function of the driving speed of the vehicle and the collection rate of the GPS receiver. For example, in the data collected for this research the test vehicle was driven at 60 mph

and the data points used were effectively collected every one second. This resulted in spacing between data points equal to roughly 88 feet.

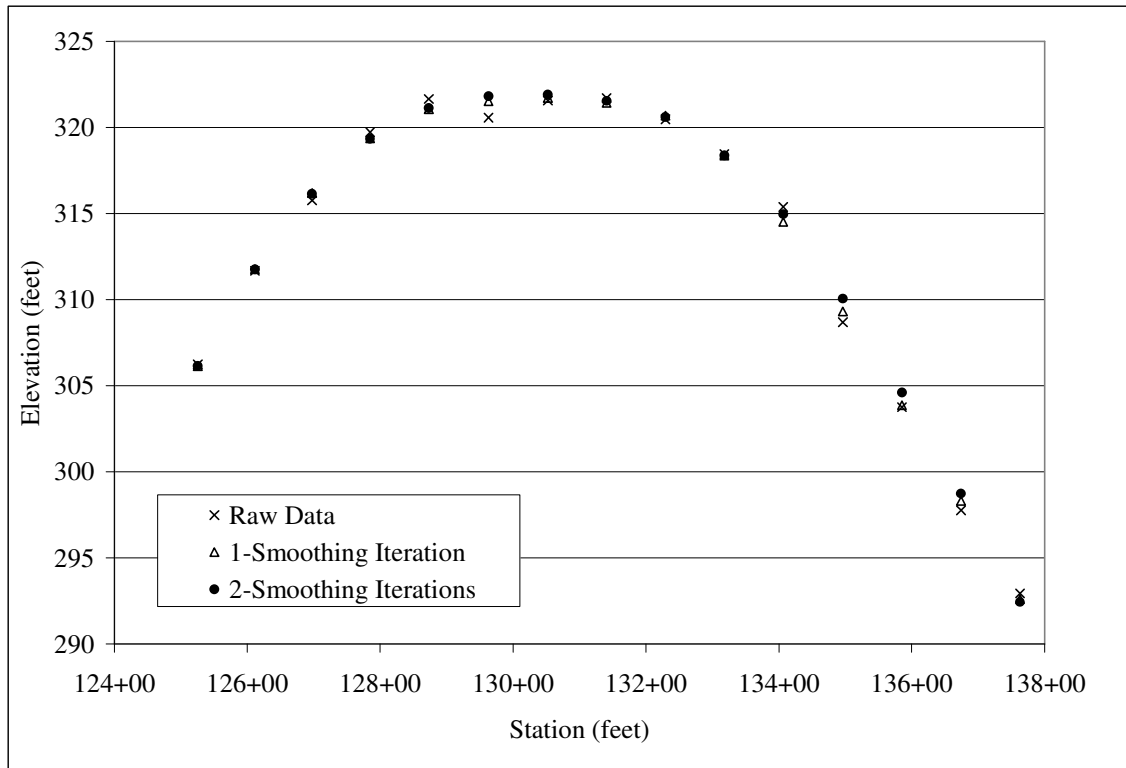


Figure IV-1. Robust Loess Smoothing Example

The no-passing zone algorithm can only make decisions based upon known or given station and elevation points. Therefore, if the GPS data is used as collected, decisions can only be made every 88 feet. Instead, by using Loess regression, data points can be estimated at much smaller intervals allowing for more frequent PSD checks in the no-passing zone algorithm.

To accomplish smaller station intervals, station values are generated every ten feet from station zero to the end of the roadway run. Then using the created station values as the test point, the same process used in smoothing is utilized; however, instead of replacing raw GPS elevations with smoothed values, completely new elevation values are calculated at the desired roadway intervals. MATLAB code from *The Data Visualization Toolbox for MATLAB* from Datatool (2007) was modified for this use. Additionally, in estimating these roadway elevations a robust method was not applied since a robust process had already been initiated in the data smoothing process. This elevation estimation process is beneficial because evenly spaced station roadway surface elevations can be calculated for input into the no-passing algorithm.

No-Passing Zone Location Algorithm

No-passing zones are based on instances where sight distance is restricted. The MUTCD defines sight distance, as needed for a pass, on a vertical curve in the following manner.

“the distance at which an object 3.5 feet above the pavement surface can be seen from a point 3.5 feet above the surface (MUTCD 2003).”

Unlike the definition used for horizontal PSD which is defined as being along the centerline of the road, the MUTCD does not specify if vertical PSD is a two-dimensional or three-dimensional distance. Since it is not defined by the MUTCD, the researcher used the distance along the centerline of the road, the same as that of the horizontal PSD.

This provides for easier and more simplified calculations in the no-passing zone algorithm.

Based on the MUTCD definition, at any point where the pavement surface restricts a driver to having sight distance less than the minimum required PSD, a no-passing zone should be established. Accordingly, the developed no-passing zone algorithm is an iterative process based on the fact that a no-passing zone will be present anytime the pavement surface elevation is above the needed sightline.

In this iterative process, there are two separate and distinct intervals. The first interval is that of testing points to determine if a no-passing zone should be identified at that location. This point will be referred to as point A. The algorithm changes the station of point A iteratively so that points every ten feet along the roadway are evaluated. The second interval pertains to the testing of each iterative location of A located every ten feet along the roadway. A secondary point, referred to as b_i , is selected 50 feet from point A. The interval between point A and point b_i is tested for sight restrictions. If a sight restriction is not found, then another interval from the next b_i , which is 50 feet further down the roadway from the current point A, whose location has not been changed, is tested for sight restrictions. Assuming no sight restrictions are found in each successive interval from A to the increasing station of b_i , the process is repeated until a distance equal to the minimum required PSD as set by the MUTCD is reached. The

point at this location will be called point C. A visual depiction of this process can be seen in Figure IV-2.

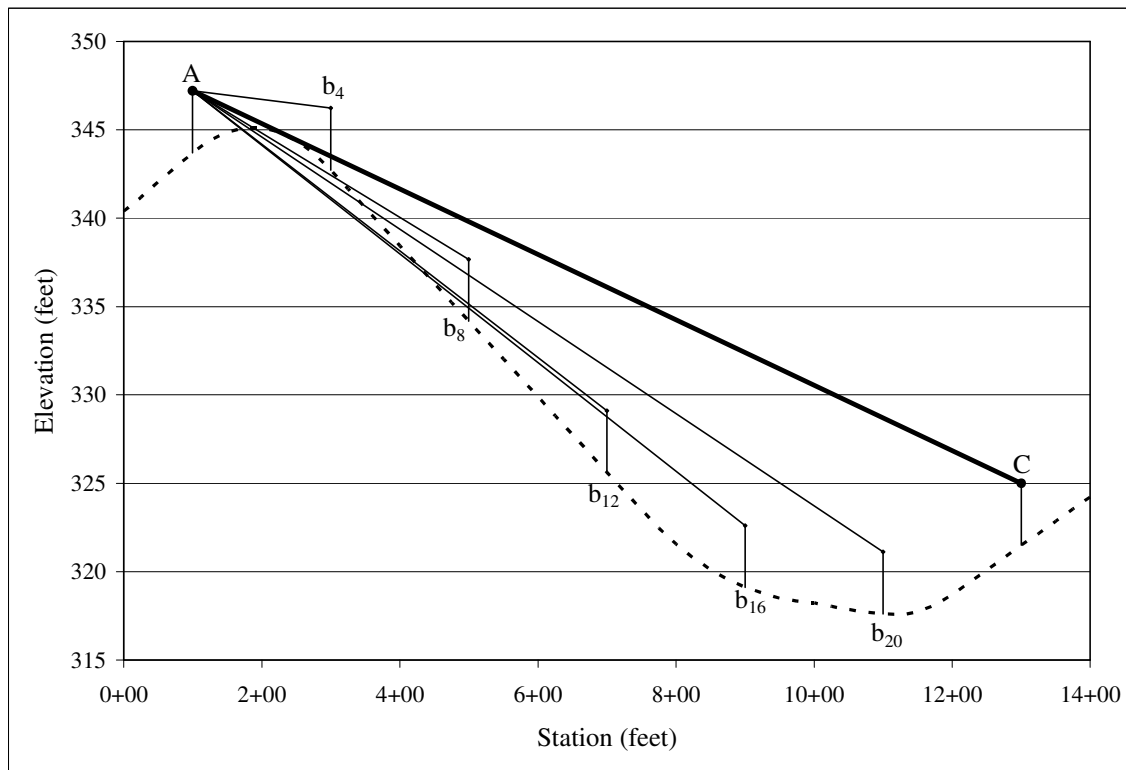


Figure IV-2. No-Passing Zone Algorithm

One reason for selecting the test interval of 50 feet in the algorithm was to reduce the required processing time. Secondly, the minimum required no-passing zone sight distances for various speeds are at intervals of 50 feet or 100 feet, so the 50 feet value fits well into the checking of the required PSD.

In general terms, the determination of the theoretical sightline that is calculated for each interval from point A to point b_i is as follows. First, 3.5 feet are added to the roadway

elevations at point A and point b_i . The line between these two points that are 3.5 feet above the pavement surface in space is the theoretical sightline. Next, the equation of the line between these two points is found where stationing is the independent variable (x) and elevation is the dependent variable (y). After determining the equation of this line, ten foot station increments are identified between point A and the ending station of the theoretical sightline at point b_i . Elevations are found for each of these stations on the sightline and compared to the corresponding roadway elevations at those precise stations as determined by the smoothing model. There are no sight restrictions in a given iteration if the sightline elevations are greater than the roadway profile. However, if at any station a roadway pavement elevation is greater than its corresponding sightline elevation, a no-passing zone exists. If this occurs, the loop of checking successive Ab_i intervals is broken. This process is repeated from the beginning of the project to the end of the project. When finished, the beginning and ending of no-passing zones are established.

As mentioned above, Figure IV-2 depicts the no-passing zone algorithm process for a roadway with a speed limit equal to 70 mph. Point A is the point in question for determining if there is a sight restriction at a given position. Point C is 1200 feet away from point A and represents the minimum required PSD as set by the MUTCD. In Figure IV-2 not all of the tested b_i intervals are shown. This is done so the general trend can be understood without cluttering the figure. Point b_4 , b_8 , b_{12} , b_{16} , b_{20} are distances of 200, 400, 600, 800, and 1000 feet respectively from point A. Figure IV-2 shows the

importance of this iterative process. If only the sight line from point A to point C is tested, the location at point A would be identified as a passing zone; however, as seen in sightlines Ab_8 , Ab_{12} , Ab_{16} , and Ab_{20} there are sight restrictions at these locations and point A should be identified as a no-passing zone location. This iterative process helps to identify possible sight valleys in the roadway.

After examining the entire roadway, the no-passing zone algorithm then identifies the beginning and ending of each no-passing zone segment. This is accomplished through a process of checking each ten foot station for which sight distances were checked to determine which points were marked as being PSD deficient. Upon identifying the no-passing zone segments the program then checks adjacent segments to see if the distance between segments is less than 400 feet. If this occurs, then the ending station of the first segment and the beginning station of the second segment are deleted and one no-passing zone is created. This follows the guidelines set forth in the MUTCD.

The final results from the no-passing zone program are output into an Excel worksheet which includes the beginning and ending stations of each no-passing zone, each no-passing zone length, the vertical profile station and smoothed elevation values, and a plot of the vertical profile for visual confirmation.

The smoothing process and no-passing zone algorithm were written in MATLAB. Upon completion, the MATLAB code was compiled into a standalone executable file that could be run outside of the MATLAB environment.

CHAPTER V

STUDY DESIGN

This chapter outlines the steps that were completed in order to test the developed no-passing zone program. The approach for locating the test sites and descriptions of each of the test sites are provided. Specific details of the data collection process concerning the number of runs completed and how the runs were conducted is included.

Information is given concerning the sensitivity analyses that were completed in addition to the different scenario and run comparisons used for analyzing the performance of the no-passing zone program. These descriptions provide the framework and background for the results presented in Chapter VI.

Testing Sites

No-passing zones caused by sight restrictions in the vertical profile were the focus of this research. As such, ideal sites for testing needed to have straight alignments and significant enough elevation changes in the vertical profile to require no-passing zones. Area Engineers in the Texas Department of Transportation (TxDOT) Bryan District were contacted and asked for recommendations of roadway segments that met these requirements. From this initial inquiry, several roadway segments were identified and these segments were driven by the researcher. After driving the recommended sites, three locations were chosen for testing and consisted of segments on Texas Farm-to-Market (FM) roads 50, 912, and 1940.

FM 912

FM 912 is in Washington County and is approximately 11 miles east on State Highway (SH) 105 from the intersection of FM 50 and SH 105. This is a two-lane roadway with no shoulders. The segment is approximately three miles long and runs from the intersection of FM 912 and SH 105 to the intersection of FM 912 and FM 1155. The horizontal alignment of this test section is straight, there are no horizontal curves, and can be seen in Figure V-1. The black dots in Figure V-1. represent the beginning and end of the roadway segment evaluated.



Figure V-1. FM 912 Alignment

FM 50

The portion of FM 50 used for testing is in Washington County, south of College Station, Texas. The actual test segment is approximately 8.5 miles long and runs from the town of Independence to the intersection of SH 105, east of Brenham, Texas. This is a two-lane roadway way that is consistently straight in the horizontal alignment with a

few horizontal curves. Figure V-2 depicts the alignment with the beginning and ending locations represented by the black dots.



Figure V-2. FM 50 Alignment

FM 1940

The last test section, FM 1940, is in Robertson County. It is approximately seven miles long and runs in a north and south direction from Old San Antonio Road to Riley Grain Road. This is again a two-lane road and except for a few isolated horizontal curves near Riley Grain Road is a straight segment. Figure V-3 provides a view of FM 1940's

alignment along with, as in the other figures, the beginning and ending positions located in figure by the black dots.

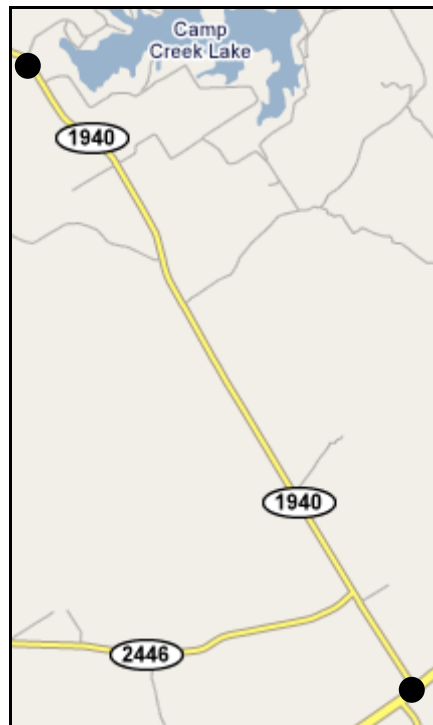


Figure V-3. FM 1940 Alignment

These three locations provided ideal locations for testing and evaluating the developed no-passing zone location system. They have significant enough elevation changes to restrict sight distances so that no-passing zones must be delineated and are straight enough that no-passing zones cannot be caused by horizontal sight obstructions, or at least the large majority cannot. The researcher did note locations on the roadways where horizontal curves potentially cause sight restrictions. The three different test locations provide several different testing lengths and consist of a variety of vertical curves.

Data Collection Methodology

At each test site, four runs were made in each travel direction of the roadway. A running start was taken with the instrumented vehicle before the starting location. This was done for two major reasons, the first being safety. By entering the traffic stream and taking a running start the instrumented vehicle did not have to sit in the middle of the road which could be hazardous since regular traffic was operating on the road. Secondly, the target speed of 60 mph could be quickly attained by taking a running start.

When the instrumented vehicle crossed the beginning point on a run, the DMI was started. A constant speed was maintained by setting the cruise control at 60 mph. Video data were collected in unison with the collection of the horizontal and vertical GPS data. This allowed for visual verification of the terrain and recording of existing pavement markings since DMI distances were superimposed on the collected video.

Software, called Dewesoft, was used in the data collection process. Along with allowing the researcher to collect video, DMI, and GPS data that were easily referenced to one another, the Dewesoft software allowed the researcher to record events at the beginning of the run, at the end of the run, and at every change between passing and no-passing zone pavement markings. Concurrently, the researcher made observations concerning whether or not the no-passing zones were the results of vertical curves only, horizontal curves only, or combinations of both.

Sensitivity Analysis Methodology

In the analysis process, the researcher examined two calculations that are involved in the no-passing zone program to see how sensitive the determinations of no-passing zones are to them. These two calculations concern station distances and the effect of vertical GPS error on PSD.

As described previously, station distances are calculated in the horizontal plane which is two-dimensional; however, in dealing with PSD as it depends on vertical roadway profiles, the distance is no longer a simple two-dimensional distance but a three-dimensional distance. The researcher examined if there is any significant difference between two-dimensional and three-dimensional distances calculated on a slope.

In examining the effect that GPS elevation inaccuracies have on PSDs, the researcher examined exclusively the possibility of inaccurate elevation readings at PSD driver and sight object locations. There are multiple scenarios which include roadway elevations at the driver and sight object locations both being below actual roadway elevations, being above and below actual roadway elevations, being below and above actual roadway elevations, or both being above actual roadway elevations respectively. Even though all of these cases are important, the last case is the focus of the sensitivity analysis because it causes the most severe threat. If this occurs, the perceived PSD will be longer than actual available PSD and could cause a no-passing zone location to be ignored.

The aforementioned analyses will be completed for each speed at which there are minimum no-passing zone PSDs. This is because each of these speeds in the Texas Roadway Design Manual (2004) have maximum slope grade values for roadway design which plays a significant part in calculations. Table V-1 below shows the speeds and their corresponding maximum slope values as seen in the Texas Roadway Design Manual (2004).

Table V-1. Design Speeds and Maximum Grades

Design Speed (mph)	Maximum Slope
25	12%
30	11%
35	10%
40	10%
45	9%
50	8%
55	8%
60	7%
65	4%
70	4%

*Values from the Texas Roadway Design Manual (2004)

The grade values from the Texas Roadway Design Manual (2004) along with the vertical curve equations from the Green Book were used to create a simple vertical curve that represents the worst possible situation at a given design speed. For example, for the design speed of 70 mph the maximum allowed grade is four percent. This value was used to create a simple symmetrical vertical crest curve with incoming and outgoing slopes of four percent. The length of the curve was found using the design equations for

a crest curve from the Green Book which utilize the algebraic difference between incoming and outgoing tangents as well as stopping sight distance. To continue the example, the algebraic difference would be eight and the stopping sight distance would have been 730 feet. Plugging these values into the correct curve length equation would produce a length of 1976 feet. Using this value a unique symmetrical vertical crest curve can be constructed for the design speed of 70 mph. Then using equations that determine the available PSD at any given point on an approaching vertical curve's tangent, the true sight distance for perfect conditions are calculated along with the sight distance associated with roadway elevations at driver and sight object locations that are above the actual conditions. Since the actual variation of the GPS elevation values is unknown, a value of 3.5 feet was used because it is equal to the driver eye and object heights. For each design speed, a unique vertical crest curve was created as describe above which represented the worst case design scenario. Comparisons were made at the point on the approaching tangent where the actual sight distance is just less than the minimum required PSD as well as at the vertical point of curvature on the vertical curve since these were found to be where the maximum and minimum passing sight distances are located in a no-passing zone.

Comparison Methodology

The GPS data will be run through the no-passing zone location program to obtain results so that several decisions may be made. One decision concerns how many runs to use in

the no-passing zone program. That is, would a single run be sufficient or would data from two, three, or four runs combined produce better results?

One problem in accomplishing this task is having an adequate measure by which to compare the various methods of combining runs. The most ideal measure would be a comparison to accurate roadway no-passing zone markings; however, the best available measures are the existing no-passing zone markings lengths as determined from the DMI values imposed on the collected video. Using the DMI and video at the speeds of 60 mph, it is not possible to identify exactly the starting and ending of the run or the starting and ending of the passing and no-passing zones of a single run. So it was difficult to obtain the absolute true location of the existing markings. Furthermore, there is no guarantee that the markings were laid out correctly. This is very important to keep in mind when viewing the results. However, by averaging DMI video results from the multiple runs in each travel direction at the test sites, the no-passing zones' lengths and locations of existing markings were obtained. These provided the basis for comparing the results of the no-passing zone algorithm presented in the next chapter. As will be seen, even this simple comparison is not easy. When processing the collected GPS values in the no-passing program, the results include several no-passing zones that when grouped together resemble more closely the existing markings. For example, when several of the algorithm identified no-passing zones are grouped together their combined length might roughly equal the measured length of one existing no-passing zone.

In light of these problems, when analyzing the data to determine the methodology for producing the best no-passing zone results the judgment was made by the researcher to group algorithm identified no-passing zones where they seemed similar to existing markings. From these values two different measures were calculated. The first were the absolute percent differences between existing no-passing zone lengths and the lengths of the group program no-passing zones. For each of the tested scenarios, an average of the absolute percent differences was taken and used for comparisons. Equation V-1 is an example of calculation steps. The second measure used was the root mean square error (RMSE) value of each of the no-passing zones in a given tested scenario. Equation V-2 shows these calculation steps. In Equations V-1 and V-2, “Existing_i” refers to no-passing zone lengths as determined from video collection data and “Program_i” refers to the grouped no-passing zone lengths.

$$\text{Average Absolute Percent Difference} = \frac{\sum_i^n \left(\frac{|\text{Existing}_i - \text{Program}_i|}{\text{Existing}_i} \right)}{n} \quad (\text{V-1})$$

$$\text{Root Mean Square Error} = \frac{\sum_i^n \sqrt{(\text{Existing} - \text{Program})^2}}{n} \quad (\text{V-2})$$

All of these measures were calculated for each of the single and combined run combinations. These results are presented in bar graph format for each of the three test sites to aide a decision concerning which single or combined run scenario to use.

Additionally, a linear graphical representation for each test site is included for visual comparison of the single and combined runs.

In combining the runs there were multiple scenarios that could be used and if all possible combinations were completed for each travel direction at each test site then the no-passing program would have to be run 90 different times. In order to reduce the number of times the no-passing zone program was run a single travel direction for each test site were selected and they are as follows: FM 912 eastbound (EB), FM 50 southbound (SB), and FM 1940 northbound (NB). Additionally, a single scenario for each of the possible single, double, triple, and quadruple run combinations were randomly selected. Those combinations are in Table V-2.

Table V-2. Tested Run Combinations

	FM 912 EB	FM 50 SB	FM 1940 NB
4 Runs	1234	1234	1234
3 Runs	234	234	123
2 Runs	13	13	23
1 Run	3	1	3

After selecting the most appropriate run combination level the researcher looked at the appropriate average absolute percent difference calculations to determine if a safety factor should be added to the no-passing zones.

CHAPTER VI

DATA ANALYSIS

In this chapter the analysis of the sensitivity of station calculations and GPS elevation inaccuracies are explored. Additionally, comparisons are made between the different run combinations at each test site. Following those evaluations, comparisons are made between the no-passing zone program results and those with an added safety factor. In all, the chapter gives a visual evaluation of the effectiveness of the program at locating no-passing zones as compared to existing marking conditions.

Sensitivity Analysis

The two areas covered in this sensitivity analysis are the effects of station calculation methodology and GPS elevation inaccuracies. More specifically, the station calculation analysis examines if there are significant differences between two-dimensional and three-dimensional station calculations. If there are, then changes may need to be implemented, but if the differences are insignificant the more simplified two-dimensional calculations can be used. The sensitivity analysis for the GPS elevation data looks at the effects inaccuracies have on PSD. The analysis examines the situation where the roadway elevations at the driver eye and object height locations are greater than the true values. Additionally, only the beginning of no-passing zones and the point of vertical curvature on a vertical curve are evaluated since these two points were found to provide the maximum and minimum PSD respectively.

Station Calculation

The first results to address in the sensitivity analysis are those associated with station calculation. Table VI-1 shows the results for each possible minimum PSD with Figure VI-1 graphically showing the results at the design speed of 25 mph.

Table VI-1. Station Calculation Methodology Comparison

Speed (mph)	Minimum PSD (ft)*	Max Slope**	Vertical Rise (ft)	Slope Dist. (ft)	Slope Dist. - Min PSD (ft)
25	450	12%	54	453.23	3.23
30	500	11%	55	503.02	3.02
35	550	10%	55	552.74	2.74
40	600	10%	60	602.99	2.99
45	700	9%	63	702.83	2.83
50	800	8%	64	802.56	2.56
55	900	8%	72	902.88	2.88
60	1000	7%	70	1002.45	2.45
65	1100	4%	44	1100.88	0.88
70	1200	4%	48	1200.96	0.96

* Minimum PSD are from the Texas MUTCD

** Max Slopes are from the Texas Roadway Design Manual

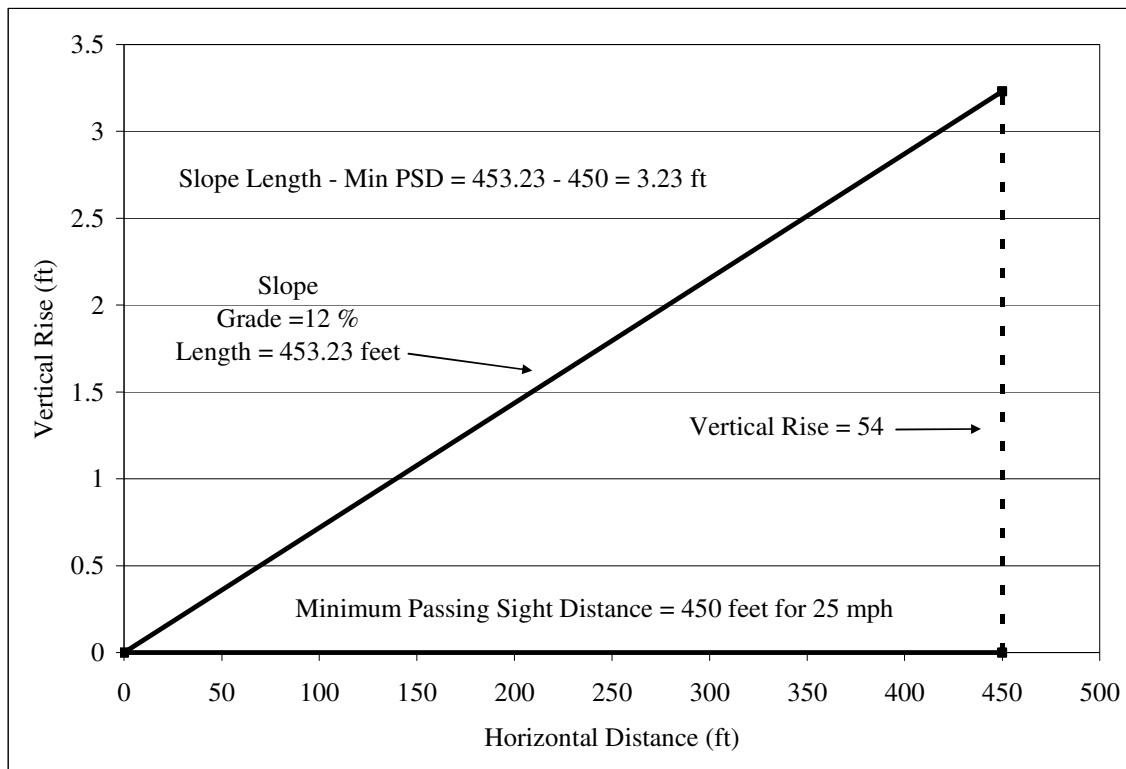


Figure VI-1. Station Calculation Methodology Comparison

The greatest algebraic difference between the two-dimensional horizontal stationing distance and the three-dimensional distance occurred at the design speed of 25 mph. The horizontal difference was 450 feet and the slope distance on a grade of 12% equaled 453.23 feet. This produced a difference of 3.23 feet which is insignificant and makes up less than one percent of 450 feet. Furthermore, the largest difference is attributed to the steepest grade, which is 12% at 25 mph. This condition will not occur often, but of possible concern is that the difference between two-dimensional and three-dimensional calculations could compound over the course of many miles. These cumulative differences might affect the form of the profile, but the differences between calculating the distances two-dimensionally or three-dimensionally were found to be negligible. For

example, on FM 912 EB using the raw run three unsmoothed data the total two-dimensional distance equaled 15,737 feet and the three-dimensional distance equaled 15,744 feet. There is only a difference of seven feet between the two station calculation methods. Furthermore, the horizontal coordinates that are used are known to have less error than the vertical elevation values so the use of only the horizontal two-dimensional coordinates makes logical sense. Additionally, the use of unsmoothed vertical data could increase the error in station calculations. The station calculation differences are not significant enough to justify changing from two-dimensional to three-dimensional station calculation.

GPS Elevation Inaccuracies

In evaluating the effects of GPS elevation inaccuracies, two conditions were checked. The first was at the location where a no-passing zone would begin and the second at the point of vertical curvature. The no-passing zone end point is not presented because it produces the same results as those of the beginning of a no-passing zone due to the use of symmetrical vertical curves. The results of the comparison between required PSD, true available PSD, and the inaccurate PSD attributed to an error at the location of the driver eye and the object height of 3.5 feet above the correct elevation are presented in Table VI-2 and Table VI-3 for the beginning of a no-passing zone and the point of vertical curvature, respectively.

Table VI-2. No-Passing Zone Beginning PSD Comparisons

Speed (mph)	Required PSD (ft)	True PSD (ft)	Inaccurate PSD (ft)	Inacc. PSD - Req. PSD (ft)	Inacc. PSD - True PSD (ft)
25	450	449	496	46	47
30	500	499	563	63	64
35	550	549	632	82	83
40	600	599	705	105	106
45	700	699	825	125	126
50	800	799	950	150	151
55	900	899	1077	177	178
60	1000	999	1208	208	209
65	1100	1099	1338	238	239
70	1200	1199	1475	275	276

Table VI-3. No-Passing Zone Vertical Point of Curvature PSD Comparisons

Speed (mph)	Required PSD (ft)	True PSD (ft)	Inaccurate PSD (ft)	Inacc. PSD - Req. PSD (ft)	Inacc. PSD - True PSD (ft)
25	450	205	271	-179	66
30	500	228	322	-178	94
35	550	285	403	-147	118
40	600	347	491	-109	144
45	700	410	580	-120	170
50	800	484	685	-115	201
55	900	564	797	-103	234
60	1000	649	918	-82	269
65	1100	735	1039	-61	304
70	1200	832	1176	-24	344

In Table VI-2 the worst results occur at the design speed of 70 mph. In this scenario the true PSD, if the roadway elevations at the driver and sight object locations are correct, is equal to 1199 feet. Alternatively, if the roadway elevation values are 3.5 feet higher

than the true values at these locations, the PSD becomes 1475 feet which is 275 feet further than the required PSD of 1200 feet. This is a severe problem. This information means that at this location a passing zone would be marked instead of a no-passing zone.

In Table VI-3 the differences between the inaccurate PSD and the required PSD are all negative which indicates that even though the inaccurate PSD are larger than the true PSD, the inaccurate PSD are still less than the minimum PSD requirements. Thus, a no-passing zone would still be established in these circumstances. This is acceptable.

What this analysis does show is the need for good GPS data that is accurate or that is at least relatively accurate. This means that the GPS elevation values do not necessarily need to be exactly equal to the true roadway elevation values; however, the elevation difference from GPS point to GPS point need to be the same as those of the true roadway profile.

The critical points in establishing no-passing zones are at the beginning and ending of the zones because at these locations the effects of inaccurate elevation data will have the most influence. The sensitivity analysis shows that at elevation differences of 3.5 feet the effects can be quite great. One option for battling this would be to simply add 275 feet to the beginning and ending of no-passing zones when the design speed of a roadway is 70 mph; however, the researcher believes this may not be the best approach. First, the elevation inaccuracies may not always be 3.5 feet. The inaccuracies could be

more or less. Additionally, different no-passing zones have different lengths; the effects of adding a uniform value to the beginning and end of a no-passing zone will be greater on a shorter no-passing zone than on a longer no-passing zone. As a result, a percentage based no-passing zone safety factor is recommended.

Run Combination Comparisons

In determining the effectiveness of the no-passing zone program the results were compared to the existing conditions. At first glance the no-passing zone program did not always produce no-passing zone segments equal to the existing conditions. Upon further evaluation, many times, the no-passing zone program had multiple no-passing zone segments that when combined produced no-passing zones that were equivalent to existing conditions. The researcher combined such no-passing zone segments in order to complete the following analysis.

Figure VI-2 represents the average absolute percent differences between the grouped no-passing zone program lengths to those of the existing conditions. The average absolute percent differences were calculated for each roadway and each scenario of a single run, two runs combined, three runs combined, and four runs combined for one direction on each of the three evaluated roadways.

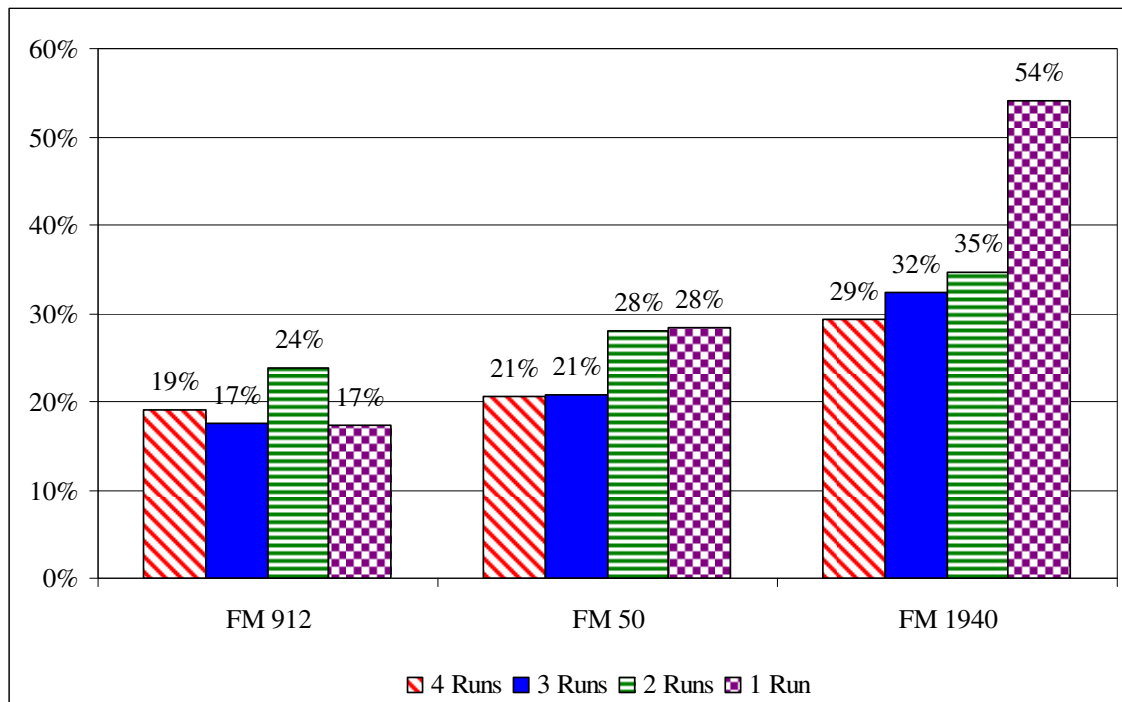


Figure VI-2. Average Absolute Percent Difference in No-Passing Zone Length

Figure VI-3 shows the RMSE for each of the tested scenarios. Though there are no clear trends in Figure VI-2 and Figure IV-3, the researcher believes the best method is that of using at least four combined runs. As shown in these figures, four combined runs are not guaranteed to out perform three combined runs or even a single individual run; the combination run method, however, does produce more stable results and helps combat the possible situation of using a single run that is very inaccurate such as the case with FM 1940.

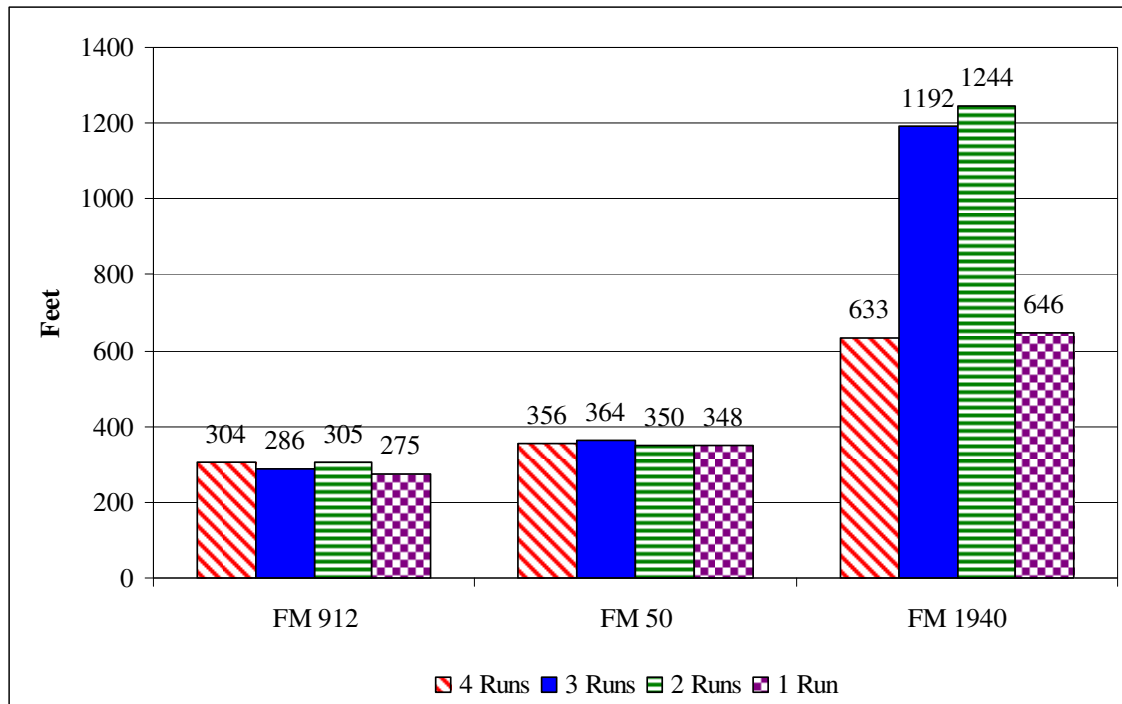


Figure VI-3. Root Mean Square Error for No-Passing Zone Lengths

For further visual confirmation, Figure VI-4 provides a visual look at the results of the various run combinations for FM 912 EB. The horizontal lines represent the lengths of the no-passing zones as determined from the no-passing zone program and from the existing conditions. The vertical lines are a guide for comparing the no-passing zone program results to the existing conditions. This graphical representation shows that the no-passing zone program is able to generally locate the existing no-passing zones, but it does come up short in comparison to existing conditions.

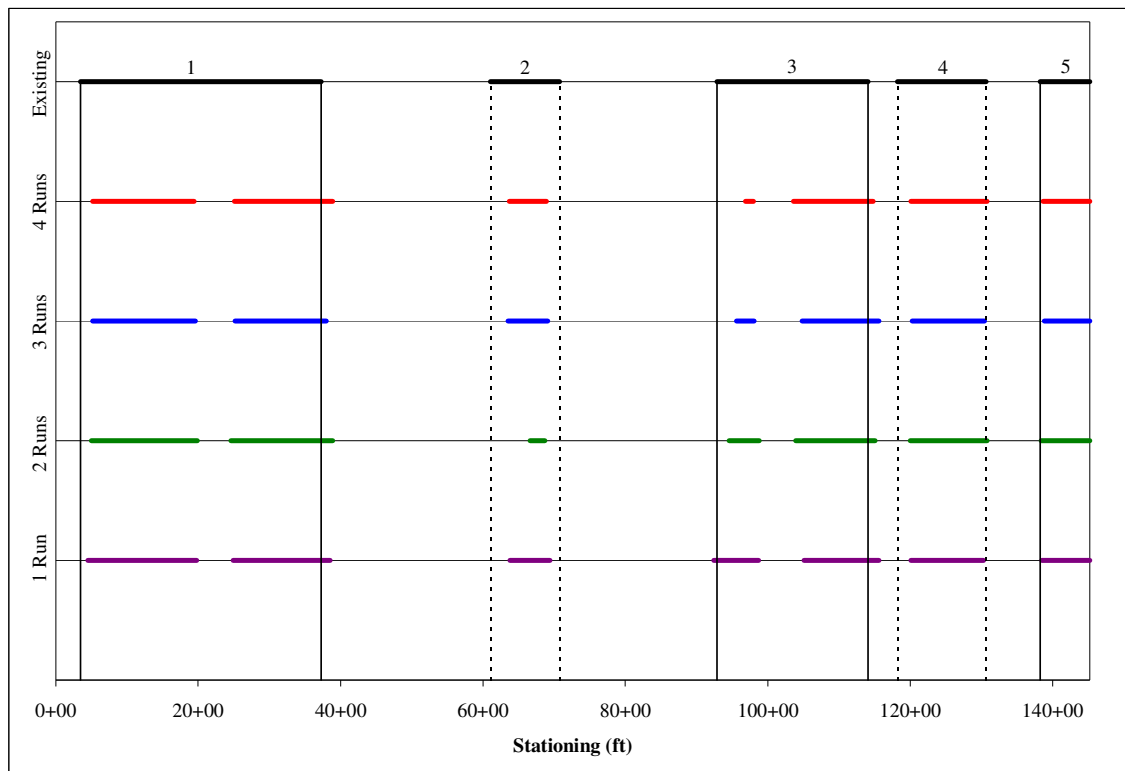


Figure VI-4. FM 912 EB

Figure VI-5 provides the same visual results for FM 50 SB. Again, the no-passing zone program is able to capture the trends of the existing conditions. One thing that Figure VI-5 shows that Figure VI-4 does not is the problem with using a single run. In Figure VI-5 on the “1 Run” line there is a no-passing zone condition that is shown between existing no-passing zones five and six. By combining multiple runs the inaccurate GPS values can be smoothed out so that this inaccurate location of a no-passing zone is eliminated.

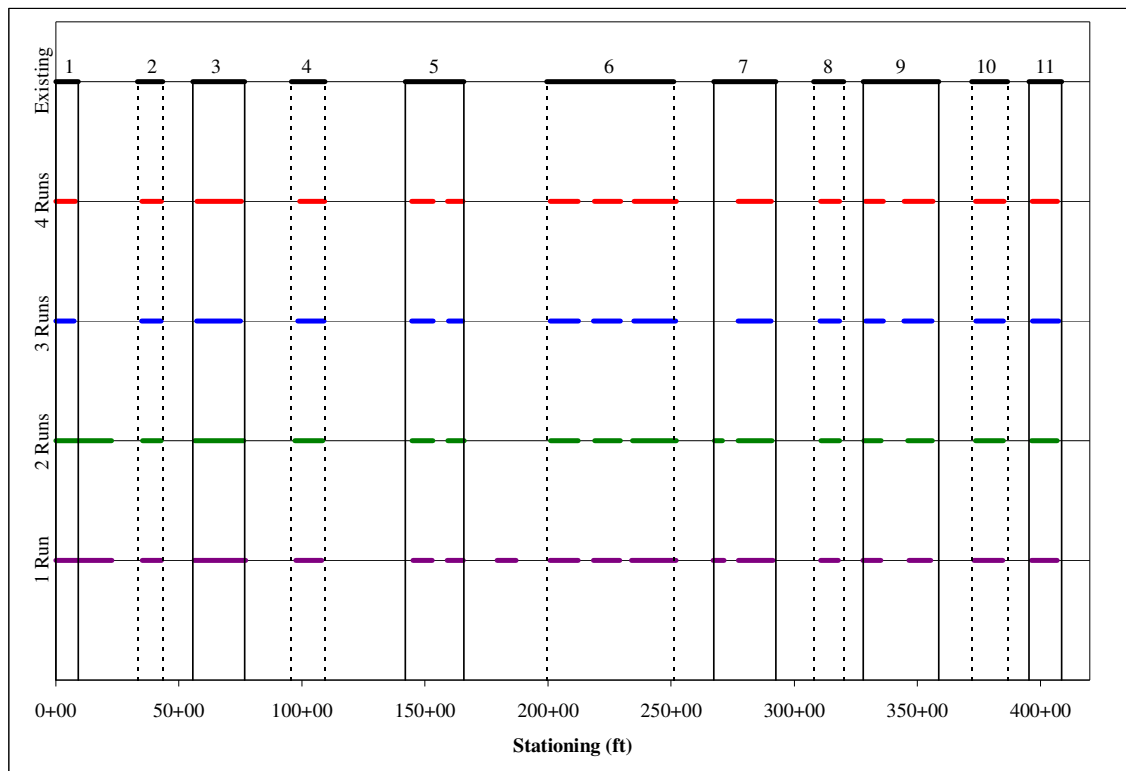


Figure VI-5. FM 50 SB

Figure VI-6 shows the results for FM 1940 NB. Even though the general locations of the existing no-passing zones are found, the results are messier than the previous two roadways. The cause of this can be traced back to the collected GPS values. When evaluating the profiles of the various runs on FM 1940 NB the variation in the GPS data can clearly be seen.

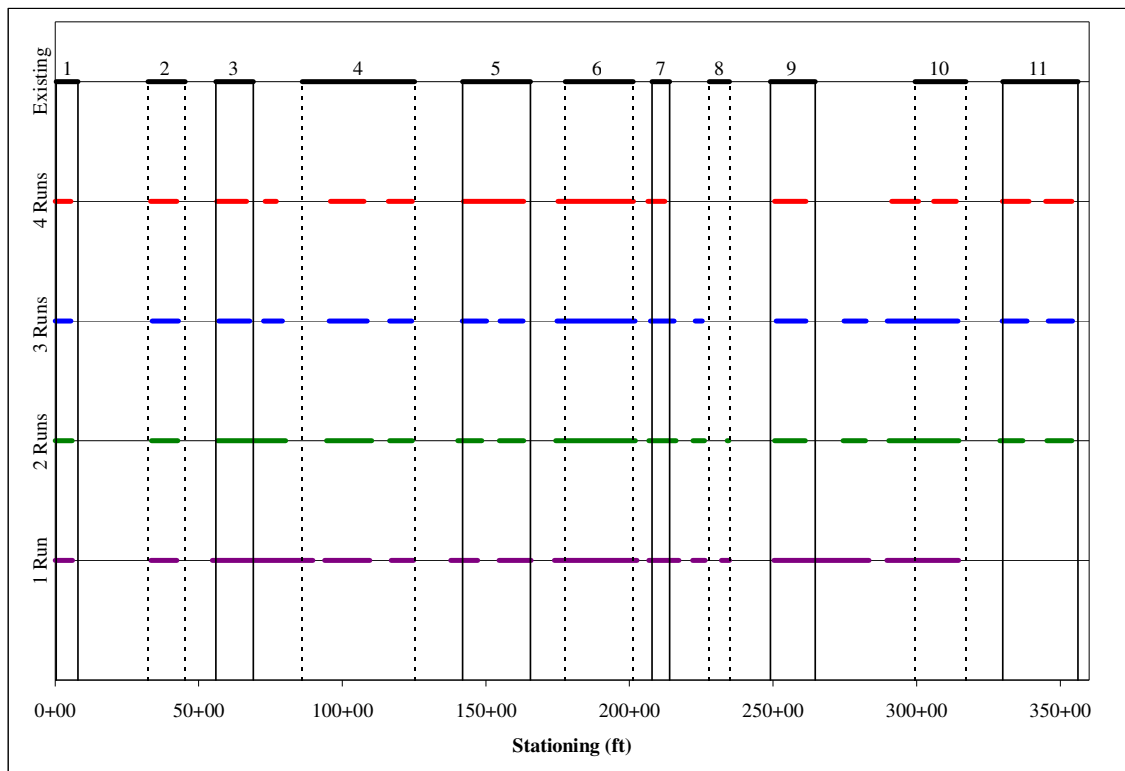


Figure VI-6. FM 1940 NB

For example, Figure VI-7 is the profile for the single run, run three, in the NB direction of FM 1940. Contrast this to Figure VI-8 which is the profile view for the single run, run three, in the EB direction of FM 912. Even though the scales are different, which can slightly distort the results; a definite difference can be seen between the consistency of the GPS data collected on FM 912 EB run three and FM 1940 NB run three. These consistency differences are directly reflected in the accuracy of the location of no-passing zones using the no-passing zone program.

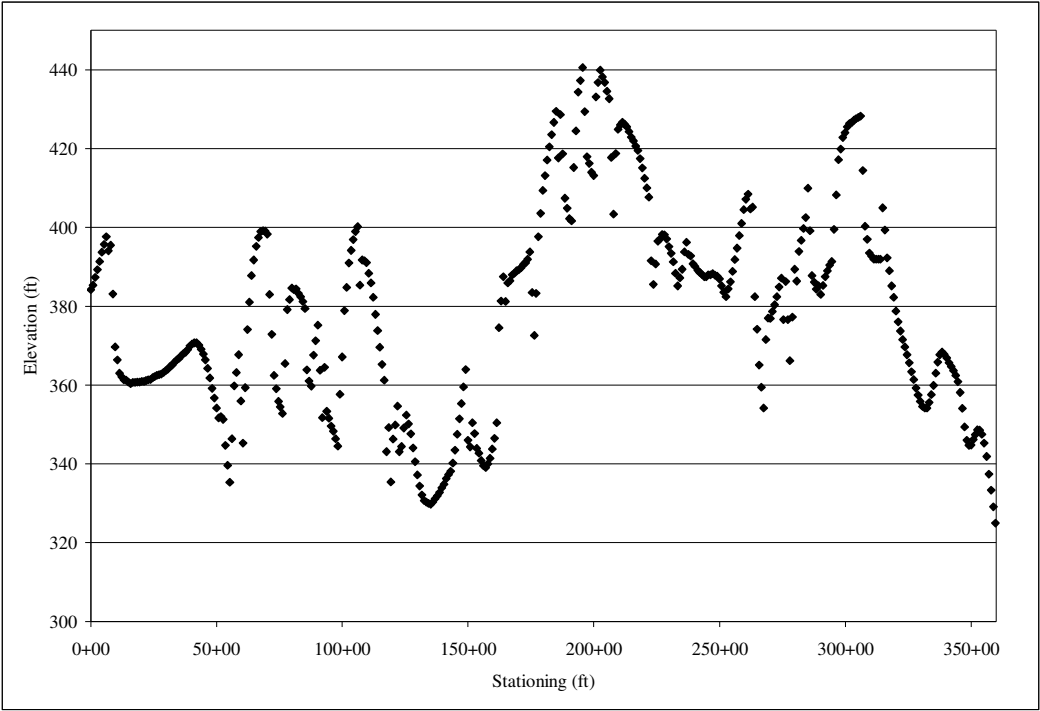


Figure VI-7. FM 1940 NB Run 3 Raw Data Profile

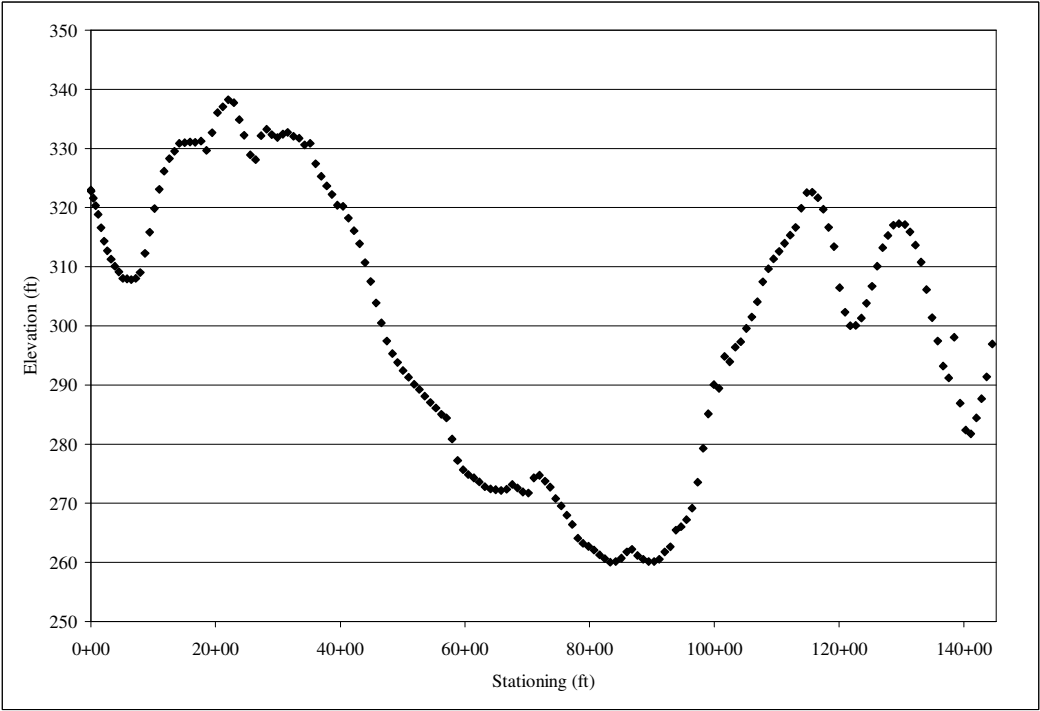


Figure VI-8. FM 912 EB Run 3 Raw Data Profile

Even though the FM 912 EB run three data is better than the FM 1940 NB data, it is still not perfect. Small jumps and gaps still occur in the data and these occurrences reinforce the need for the smoothing processes. Figure VI-9 is the FM 912 EB run three profile after the data has been smoothed and the estimated ten foot interval station elevations calculated. Figure VI-9 can be directly compared to Figure VI-10, which is the vertical profile of FM 912 EB as taken from design sheets, to see the effectiveness of the smoothing and estimation process.

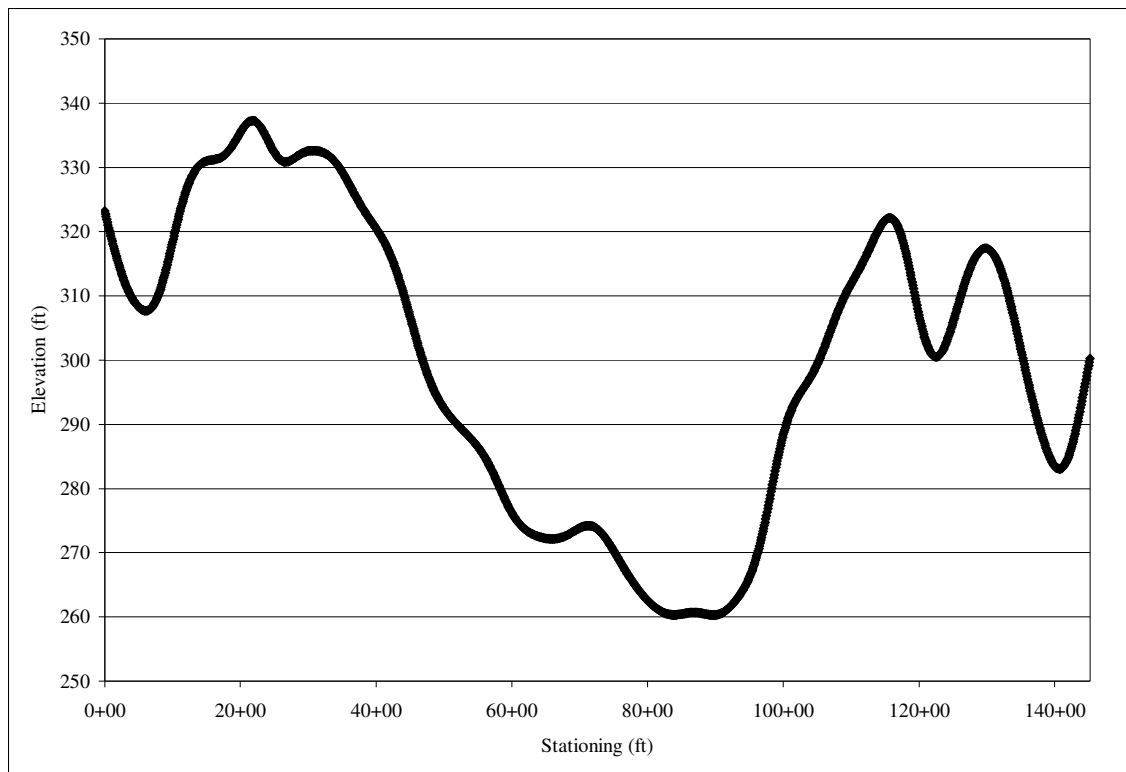


Figure VI-9. FM 912 EB Smoothed Profile

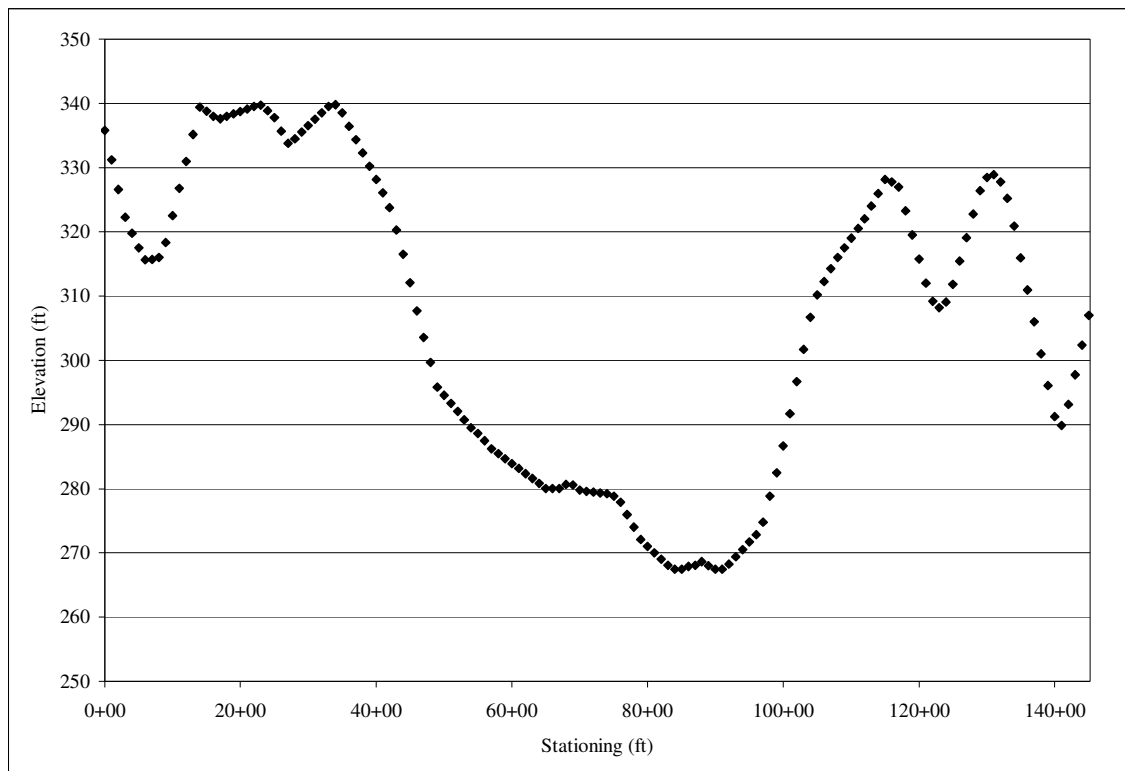


Figure VI-10. FM 912 EB Design Profile

Safety Factor Comparisons

As mentioned earlier, the researcher recommended a percentage based safety factor for the no-passing zones as a possible approach. When the average absolute percent differences in no-passing zone lengths for the three roads on which four different runs were combined, they ranged from basically 20 to 30 percent. Based on these numbers the researcher concluded that a 25 percent safety factor would be sufficient with 12.5 percent of a no-passing zone length being added to both the beginning and ending of the no-passing zones as established by the no-passing zone program. After adding the safety factor distances onto the determined no-passing zones, the no-passing zones can then be

revaluated, and adjacent no-passing zones that are less than 400 feet apart can be combined.

After adding in the safety factors and then grouping the no-passing zones as was done in the previous analysis, Figure VI-11 Figure VI-12 show the average absolute percent difference in no-passing zone lengths and the RMSE in no-passing zone lengths. The graphs show that the safety factor does help improve the results, but at the same time the safety factor does not completely remedy the problem since the averages of the absolute percent differences and RMSE values are not zero.

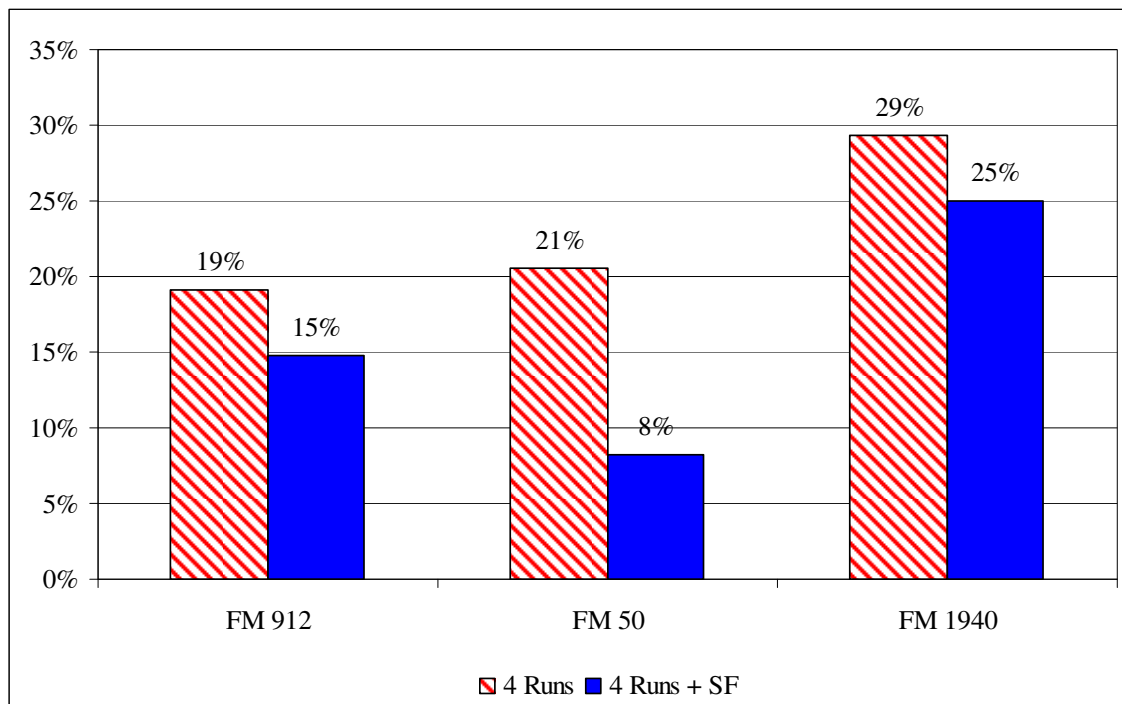


Figure VI-11. Average Absolute Percent Difference Safety Factor Comparison

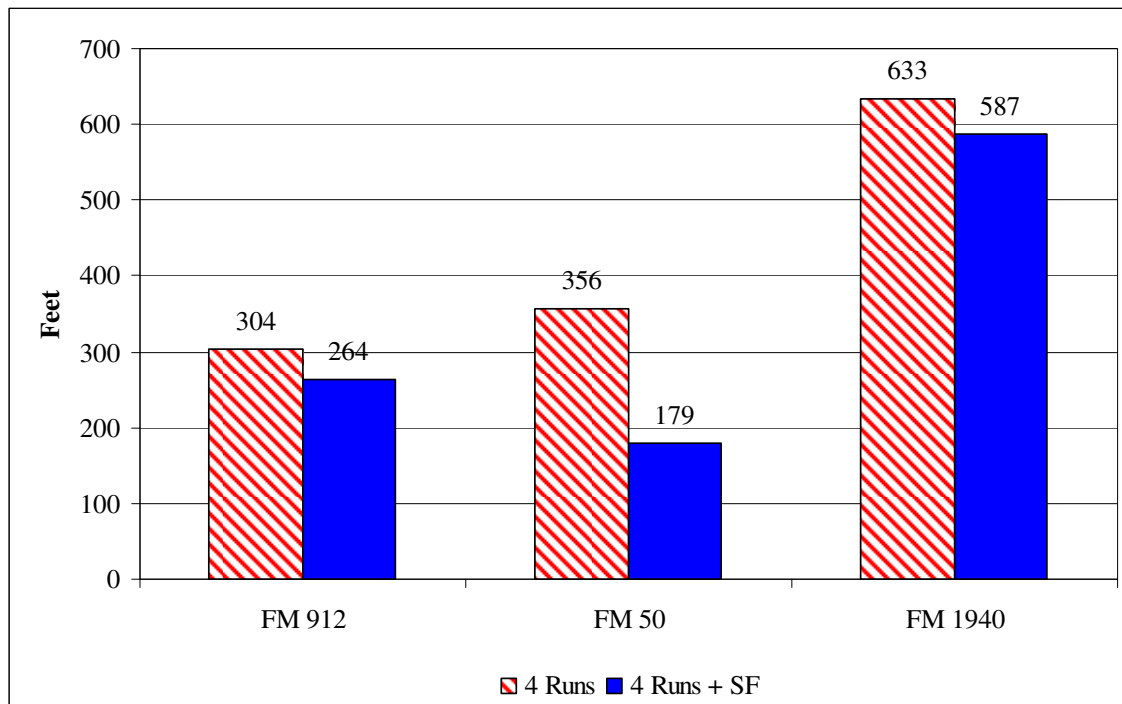


Figure VI-12. Root Mean Square Error Safety Factor Comparison

Figure VI-13 gives a visual comparison of the no-passing zones as found by the original no-passing zone program and those after the safety factor is added for FM 912. A major improvement is seen in no-passing zone one in that the two segments as found by the original results are connected and extended. This shows the added benefit of the safety factor; however, even though the safety factor does help extend the no-passing zones, some still come up short as seen in no-passing zone two and at the beginning of no-passing zone three.

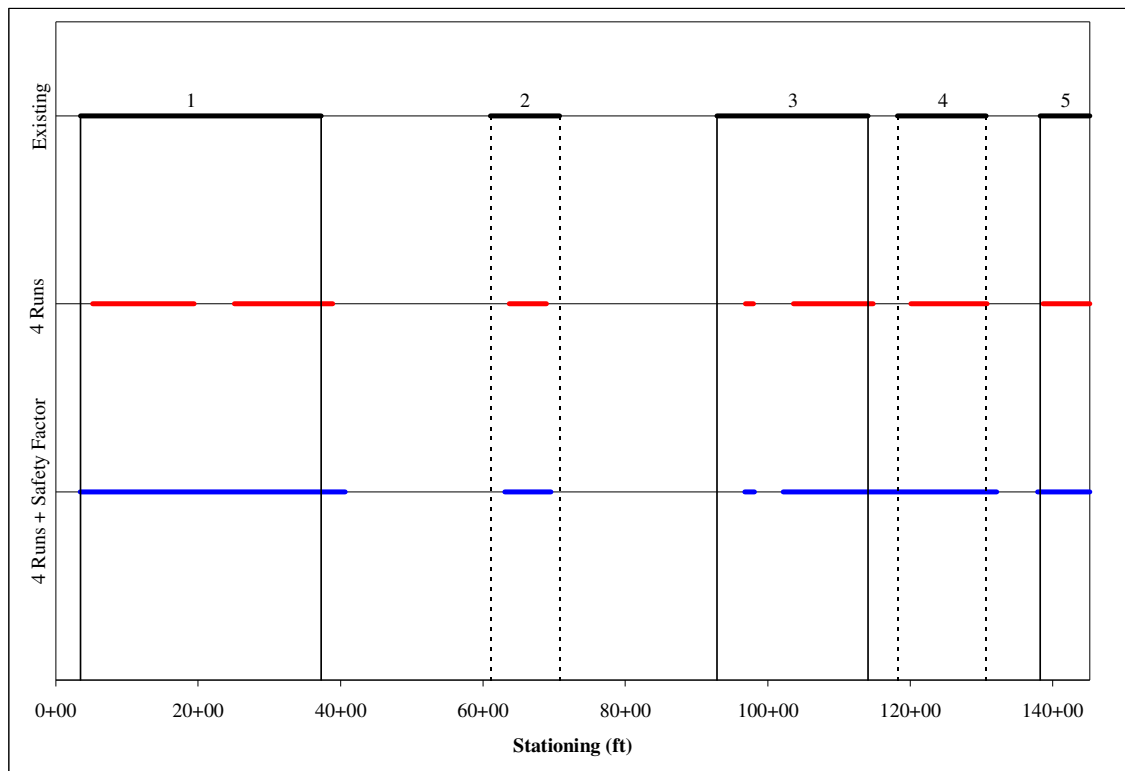


Figure VI-13. FM 912 EB Safety Factor Comparison

Figure VI-14 is a graphical comparison for FM 50 SB. The benefits of a 25 percent no-passing zone length safety factor can be seen in no-passing zone three where the beginning and ending locations are extended before and after those of the existing conditions. This is acceptable. Further benefits of the safety factor are seen in no-passing zone six. The original program no-passing zone segments have been combined to create a single no-passing zone at this location, but as was seen on FM 912 EB, there are situations where the safety factor still comes up short in its effectiveness. For example, in no-passing zones seven and eight the no-passing zone beginning locations do not extend to those of the existing conditions. These are troubling.

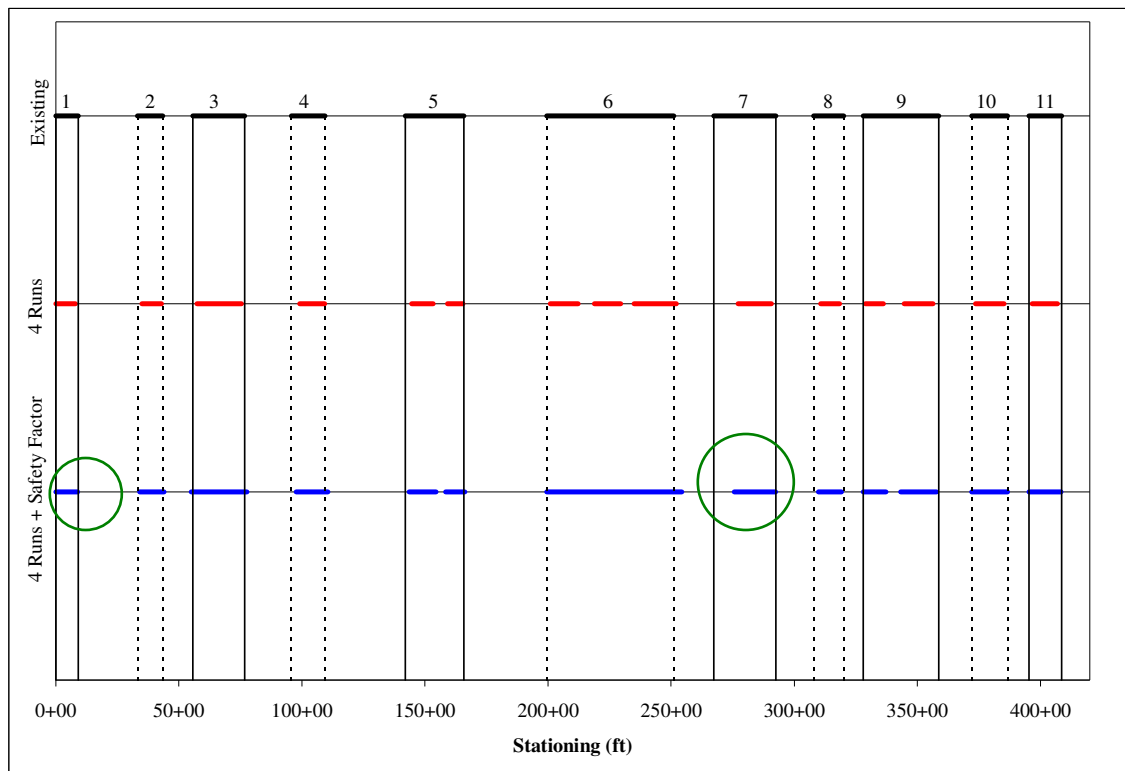


Figure VI-14. FM 50 SB Safety Factor Comparison

One possible consolation is that on FM 50 the researcher noted at least two significant horizontal curves while driving the roadway that could affect sight distance. Those two horizontal curves occurred in no-passing zones one and seven as identified by the circles in Figure VI-14. No-passing zone one does not seem to be affected by the horizontal curves. As for the seventh no-passing zone, the researcher is of the opinion that the horizontal curve does not have a significant impact because of the nature of the vertical geometry of the roadway at this location. However, it is difficult to quantify the effect of the horizontal curves because they occurred in conjunction with vertical curves. Since the no-passing algorithm currently only adjusts for no-passing zones caused by

vertical alignments, these instances need to be noted because they might possibly cause some of the mentioned deficiencies.

Figure VI-15 shows the safety factor comparisons for FM 1940 NB. The effects of the safety factor, although good, are not as prominent on FM 1940 as they were on the previous two roadways. The conclusion for this goes back to inaccurate and jumpy GPS data. The researcher believes that if better GPS data were obtained for FM 1940 then better results could be produced.

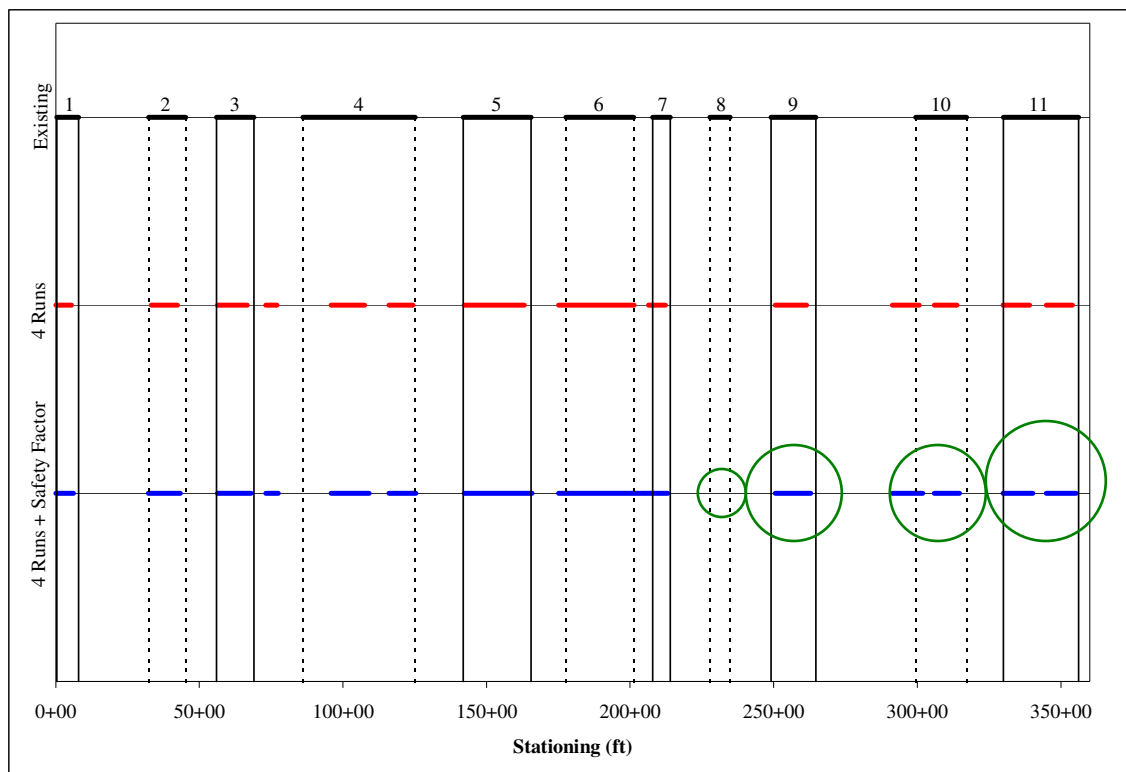


Figure VI-15. FM 1940 NB Safety Factor Comparison

In the NB direction of FM 1940 there were several no-passing zones in which vertical as well as horizontal curves occurred. These occurred in existing marked no-passing zones eight, nine, ten, and eleven which are circled in Figure VI-15. In no-passing zones eight, nine, and ten the horizontal curves occurred near the end of the existing no-passing zone, while the horizontal curve occurred near the middle of existing no-passing zone eleven. Again, determining how much, if any, of the existing no-passing zones are a result of horizontal sight restrictions is difficult because the horizontal curves occur in conjunction with the vertical curves on the roadway. However, the instances of these horizontal curves actually coordinate with the location of some of the gaps seen in Figure VI-15.

CHAPTER VII

CONCLUSIONS

This chapter summarizes the work completed by the research, gives some conclusions and recommendations from the researcher, and provides tasks for future research.

Although the research may not be implementable at the current time, it does provide the basis for an automated system. If the future research recommendations are completed, the possibility of having a product that can be used by field crews in marking no-passing zones is believed feasible.

Conclusions

The research presented in this thesis explored a new automated method for locating no-passing zones in the field using GPS. Processes for collecting GPS data, converting GPS data, and smoothing GPS elevation data were presented. Additionally, an algorithm for evaluating the smoothed GPS data for no-passing zone locations was created. The program, as seen in chapter VI, captured the general locations of the no-passing zones. Upon further evaluation a 25 percent safety factor was added in order to improve the capabilities of the no-passing zone program as compared to existing measured field no-passing zone markings. However, even though the safety factor improved the results, it still came up short in completely matching the existing marking conditions.

The researcher's conclusion is that the developed system shows promise in being able to automatically locate no-passing zones using GPS data. This is especially true when multiple roadway runs are conducted and combined. Despite these promises, the system is not ready for implementation; instead, further research should be conducted.

Future Work

There are two problems that should be the focus of future research. The first is the measure by which the no-passing zone results were compared. In the analysis, the no-passing zone results were compared to existing no-passing zone markings, but in doing so the researcher notes two issues. The first has to do with the video and DMI method by which the existing markings were measured. This method was not perfect by any means, but gave the researcher a ballpark measurement by which to compare results. The second issue is that there is no guarantee that the existing no-passing zones are correct. The researcher operated on the assumption that they were indeed correct. In order to remedy this problem, one or all of the tested roadways should be walked by a crew using the walking method of locating no-passing zones. This would most likely be the best method, although a slow one, for accurately establishing no-passing zone locations. Then true comparisons could be made between the no-passing zone program results and true no-passing zone locations.

The second problem that should be the focus of future research is the accuracy of collected GPS data. In order to confirm this, at least one of the test roadways needs to

be surveyed by a surveyor in order to obtain an accurate roadway profile. Elevations should be gathered every ten feet along the roadway surface. This interval is recommended for two reasons, the first being that this is the interval the no-passing zone program tests for PSD restrictions. These surveyed ten foot stations could then be directly plugged into a modified no-passing zone program to produce results that could be directly compared to those found by the walking method alluded to above. This would provide a check for the methodology of the no-passing zone program algorithm. Although, the researcher feels that the proposed methodology is sound, a further check such as this would be beneficial. The second reason for surveying the roadway at ten foot intervals is for comparison of the smoothed Loess Regression vertical profile to true accurate data. If ten foot station profile elevations were known, then direct comparison could be made between the smoothed GPS profile as determined from the collected data of the moving vehicle.

In addition to research which addresses these two problems, an extensive analysis should be completed of the available GPS technologies. This was not done in this research and is an essential factor that must be explored before an actual system can be implemented. Such research would go hand in hand with the surveying of a road since adequate GPS data would be available as a check.

Additionally, software for automatically retrieving data from a GPS receiver and that coordinates the various preprocessing steps created by the research is needed. This

technology may be easily achievable but necessary resources were not available to the researcher at this time.

By accomplishing these tasks, the legitimacy of the no-passing zone program could be further tested and refined so that a completely automated no-passing zone location system that is accurate, efficient, and safe could be developed and implemented.

REFERENCES

AASHTO (2004). *A Policy on Geometric Design of Roadways and Streets*, 5th Edition, Washington, D.C.

Ben-Arieh, D., Chang, S., Rys, M., and Zhang, G. (2004). “Geometric Modeling of Roadways Using Global Positioning System Data and B-Spline Approximation.” *Journal of Transportation Engineering*, 130(5), 632-636.

Brown, R.L., and Hummer, J.E. (2000). “Determining the Best Method for Measuring No-Passing Zones”. *Transportation Research Record 1701*, Transportation Research Board, Washington, D.C.

Chang, S.I. (2004). “Global Positioning System Data Integration and Development of a Three-Dimensional Spatial Model of the Kansas Highway Network.” FHWA-KS-03-7, Kansas Department of Transportation, Topeka, Kansas.

Chapra, S.C. and Canale, R.P. (2002). *Numerical Methods for Engineers with Software and Programming Applications*, 4th Ed., McGraw Hill, Boston.

Cleveland, W.S. (1993). *Visualizing Data*. AT&T Bell Laboratories, Murray Hill, New Jersey.

Datatool. *The Data Visualization Toolbox for MATLAB*. Online www.datatool.com. Accessed November 2007.

Easa, S.M. (1999). "Optimum Vertical Curves for Roadway Profiles." *Journal of Surveying Engineering*, 125(3), 147-157.

Easa, S.M., Hassan, Y., and Karim, Z.A. (1998). "Establishing Highwayway Vertical Alignment Using Profile Field Data." *ITE Journal on the Web*, <http://www.ite.org/itejournal/webarticles.asp>, Accessed April 2008.

El-Rabbany, A. (2006). *Introduction to GPS: The Global Positioning System*, 2nd Ed., Artech House, Boston.

Fitzpatrick, K., Parham, A.H., Brewer, M.A., and Miaou, S.-P. (2001). *Characteristics of Potential Treatments for Crashes on Low-Volume, Rural Two-Lane Roadways in Texas*. FHWA/TX-02/4048-1, Texas Transportation Institute, The Texas A&M University System, College Station, Texas.

Hardwood, D. W., Gilmore, D.K., Potts, I.B., Torbic, D.J., and Council, F.M. (2005). "Passing Sight Distance Criteria". NCHRP Project 15-26, Transportation Research Board, <http://www.trb.org/NotesDocs/NCHRP15-26IR.pdf>, Accessed February 2007.

MATLAB Support Page: Spline Toolbox. The Mathworks, Inc.,
www.mathworks.com/support/. Accessed September 26, 2007.

Nehate, G., and Rys, M. (2006). "3D Calculation of Stopping-Sight Distance from GPS Data." *Journal of Transportation Engineering*, 132(9), 691-698.

Namala, S.R., and Rys, M. (2006). "Automated Calculation of Passing Sight Distance Using Global Positioning System Data." *Kansas DOT Rep. No. K-TRAN: KSU-03-02*, Kansas Department of Transportation, Topeka, Kansas.

MUTCD (*Manual on Traffic Control Devices*) (2003). Federal Roadway Administration, Washington, D.C.

Texas Roadway Design Manual (2004). Texas Department of Transportation, Austin, Texas.

Rose, E.R., Hawkins, H.G., Holick, A.J., and Bligh, R.P. (2004). *Evaluation of Traffic Control Devices: First Year Activities*. FHWA/TX-05-0-4701-1, Texas Transportation Institute, The Texas A&M University System, College Station, Texas.

Texas Centric Mapping System/Lambert Conformal. I Texas Administrative Code, Part 10, Section 201.6.

Texas Manual on Traffic Control Devices, 2006 Edition (2006). Texas Department of Transportation, Austin, Texas.

Traffic Control Devices Handbook (2001). Publication No. IR-112, Institute of Transportation Engineers, Washington, D.C.

Wolf, P.R. and Ghilani, C.D. (2006). *Elementary Surveying: An Introduction to Geomatics*. Pearson Prentice Hall, Upper Saddle River, New Jersey.

Young, S.E. (2004). “Development of a Contiguous Three-Dimensional Geometric Roadway Model Using Historical Global Positioning Data.” Ph.D Dissertation, Kansas State University, Manhattan, Kansas.

APPENDIX

Excel VBA Easting Function Code

(Lines with ' in front of them are comments and not executable code)

Function Easting(latdeg, longdeg)

'This function converts latitudes and longitudes that are in degrees decimal degrees into
'Eastings that are in U.S. survey feet for the Texas Centric Mapping System/Lambert 'Conformal
(TCMS/LC).

'TCMS/LC uses the North American Datum of 1983 (NAD83) and the GRS-80 'Ellipsoid.

'*Very IMPORTANT* --> the latitude & longitudes must use the same Datum & 'Ellipsoid

'WGS-84 Equivalent to NAD-83 in North America

'OmniSTAR VBS correction service uses NAD-83 Datum and GRS-80 Ellipsoid

'WAAS correction service uses WGS-84 Datum and Ellipsoid

a = 6378137

'Semiaxis a for GRS-80 Ellipsoid, units in meters

b = 6356752.3

'Semiaxis b for GRS-80 Ellipsoid, units in meters

latitude0 = Application.WorksheetFunction.Radians(18)

'Latitude of Origin for TCMS/LC converted to Radians (North)

longitude0 = Application.WorksheetFunction.Radians(-100)

'Longitude of Origin for TCMS/LC converted to Radians (West)

latitudeS = Application.WorksheetFunction.Radians(27.5)

'Lower Standard Parallel for TCMS/LC converted to Radians (North)

latitudeN = Application.WorksheetFunction.Radians(35)

'Upper Standard Parallel for TCMS/LC Converted to Radians (North)

Eo = 1500000

'False Easting for TCMS/LC, units in meters

No = 5000000

'False Northing for TCMS/LC, units in meters

latdeg = Application.WorksheetFunction.Radians(latdeg)

'Input latitude in Degrees decimal degrees

longdeg = Application.WorksheetFunction.Radians(longdeg)

'Input longitude in Degrees decimal degrees

f = 1 - (b / a)

'Flattening f of the ellipsoid for GRS 80

'f = 1/298.257223563

'Flattening f of the ellipsoid for WGS 84

e = Sqr(2 * f - f ^ 2)

'First eccentricity e of the ellipsoid

w1 = Sqr(1 - (e ^ 2) * Sin(latitudeS) ^ 2)

w2 = Sqr(1 - (e ^ 2) * Sin(latitudeN) ^ 2)

m1 = Cos(latitudeS) / w1

$$m2 = \text{Cos}(\text{latitudeN}) / w2$$

$$t_0 = \text{Sqr}(((1 - \text{Sin}(\text{latitude0})) / (1 + \text{Sin}(\text{latitude0}))) * ((1 + e * \text{Sin}(\text{latitude0})) / (1 - e * \text{Sin}(\text{latitude0})))) ^ e)$$

$$t_1 = \text{Sqr}(((1 - \text{Sin}(\text{latitudeS})) / (1 + \text{Sin}(\text{latitudeS}))) * ((1 + e * \text{Sin}(\text{latitudeS})) / (1 - e * \text{Sin}(\text{latitudeS})))) ^ e)$$

$$t_2 = \text{Sqr}(((1 - \text{Sin}(\text{latitudeN})) / (1 + \text{Sin}(\text{latitudeN}))) * ((1 + e * \text{Sin}(\text{latitudeN})) / (1 - e * \text{Sin}(\text{latitudeN})))) ^ e)$$

$$n = (\text{Log}(m1) - \text{Log}(m2)) / (\text{Log}(t_1) - \text{Log}(t_2))$$

$$F2 = m1 / (n * (t_1 ^ n))$$

$$Rb = a * F2 * (t_0 ^ n)$$

$$y = (\text{longitude0} - \text{longdeg}) * n$$

$$t = \text{Sqr}(((1 - \text{Sin}(\text{latdeg})) / (1 + \text{Sin}(\text{latdeg}))) * ((1 + e * \text{Sin}(\text{latdeg})) / (1 - e * \text{Sin}(\text{latdeg})))) ^ e)$$

$$R = a * F2 * (t ^ n)$$

$$\text{'longtox} = R * \text{Sin}(y) + Eo$$

'Easting units in meters

$$\text{Easting} = (R * \text{Sin}(y) + Eo) * 39.37 / 12$$

'Easting U.S. Survey Feet

End Function

Excel VBA Northing Function Code

(Lines with ' in front of them are comments and not executable code)

Function Northing(latdeg, longdeg)

'This function converts latitudes and longitudes that are in decimal degrees into
'Northings that are in U.S. survey feet for the Texas Centric Mapping System/Lambert 'Conformal
'(TCMS/LC).

'TCMS/LC uses the North American Datum of 1983 (NAD83) and the GRS-80 'Ellipsoid.

'*Very IMPORTANT* --> the latitude & longitudes must use the same Datum & 'Ellipsoid

'WGS-84 Equivalent to NAD-83 in North America

'OmniSTAR VBS correction service uses NAD-83 Datum and GRS-80 Ellipsoid

'WAAS correction service uses WGS-84 Datum and Ellipsoid

a = 6378137

'Semiaxis a for GRS-80 Ellipsoid, units in meters

b = 6356752.3

'Semiaxis b for GRS-8- Ellipsoid, units in meters

latitude0 = Application.WorksheetFunction.Radians(18)

'Latitude of Origin for TCMS/LC converted to Radians (North)

longitude0 = Application.WorksheetFunction.Radians(-100)

'Longitude of Origin for TCMS/LC converted to Radians (West)

latitudeS = Application.WorksheetFunction.Radians(27.5)

'Lower Standard Parallel for TCMS/LC converted to Radians (North)

latitudeN = Application.WorksheetFunction.Radians(35)

'Upper Standard Parallel for TCMS/LC Converted to Radians (North)

Eo = 1500000

'False Easting for TCMS/LC, units in meters

No = 5000000

'False Northing for TCMS/LC, units in meters

latdeg = Application.WorksheetFunction.Radians(latdeg)

'Input latitude in Degrees decimal degrees

longdeg = Application.WorksheetFunction.Radians(longdeg)

'Input longitude in Degrees decimal degrees

f = 1 - (b / a)

'Flattening f of the ellipsoid for GRS 80

'f = 1/298.257223563

'Flattening f of the ellipsoid for WGS 84

e = Sqr(2 * f - f ^ 2)

'First eccentricity e of the ellipsoid

w1 = Sqr(1 - (e ^ 2) * Sin(latitudeS) ^ 2)

w2 = Sqr(1 - (e ^ 2) * Sin(latitudeN) ^ 2)

m1 = Cos(latitudeS) / w1

m2 = Cos(latitudeN) / w2


```
t_0 = Sqr(((1 - Sin(latitude0)) / (1 + Sin(latitude0))) * ((1 + e * Sin(latitude0)) / (1 - e * Sin(latitude0))) ^ e)
```

```
t_1 = Sqr(((1 - Sin(latitudeS)) / (1 + Sin(latitudeS))) * ((1 + e * Sin(latitudeS)) / (1 - e * Sin(latitudeS))) ^ e)
```

```
t_2 = Sqr(((1 - Sin(latitudeN)) / (1 + Sin(latitudeN))) * ((1 + e * Sin(latitudeN)) / (1 - e * Sin(latitudeN))) ^ e)
```

```
n = (Log(m1) - Log(m2)) / (Log(t_1) - Log(t_2))
```

```
F2 = m1 / (n * (t_1 ^ n))
```

```
Rb = a * F2 * (t_0 ^ n)
```

```
y = (longitude0 - longdeg) * n
```

```
t = Sqr(((1 - Sin(latdeg)) / (1 + Sin(latdeg))) * ((1 + e * Sin(latdeg)) / (1 - e * Sin(latdeg))) ^ e)
```

```
R = a * F2 * (t ^ n)
```

```
'lattoy = Rb - R * Cos(y) + No
```

```
'Northing in meters
```

```
Northing = (Rb - R * Cos(y) + No) * 39.37 / 12
```

```
'Northing in U.S. Survey Feet
```

```
End Function
```

Excel VBA DeltaDist Function Code

```
Function DeltaDist(x1, x2, y1, y2)
DeltaDist = Sqr((x2 - x1) ^ 2 + (y2 - y1) ^ 2)
End Function
```

No-Passing Zone Program MATLAB Code

```
function NoPassingLoess()

datain=xlsread('NPZinput.xls');
% This file should contain two columns, column one is raw station/distance data and column two is raw
% elevation data. There are no headings
s1=datain(:,1);
% Extract raw station data and place in column array s1
z1=datain(:,2);
% Extract raw elevation data and place in column array z1

speed = datain(1,3);
% speed is the design speed or 85 percentile speed of the roadway under evaluation
spanlength = datain(1,4);

%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%

A = [s1 z1];
% Combine station and elevation data into one matrix
B = sortrows(A,1);
% Sort the data in ascending order according to stations, elevations stay with original stations
s = B(:,1);
% Extract station data from the combined sorted B matrix
z = B(:,2);
% Extract elevation data from the combined sorted B matrix

station_difference = diff(s);
% Calculate the differences between station data points
avgdiff = mean(station_difference);
% Calculate the average difference between station data points
span = spanlength/avgdiff;
% Calculate how many points are to be evaluated in the loess regression smoothing process
span1 = round(span);
% Round the span to the nearest integer

%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%

z1 = smooth(s,z,span1,'rloess');
% First smoothing run using the robust loess regression: s is station array, z is elevation array, span1 is
% span of points evaluated in loess regression intervals
z3 = smooth(s,z1,span1,'rloess');
% Second smoothing run, considers a span of 20 points
%z3 = smooth(s,z2,span,'rloess');
% Third smoothing run, considers a span of 10 points
```

[illegible]

```

i = 0;
% Index for looping through elevation matrix
continue2 = 1;
% Condition for continuing to loop through elevation matrix, 1 = true, 0 = false

while continue2 == 1
% As long as continue2 is true, keep going through loop
    i = i + 1;
    j = i + 1;
    k = j + 1;
    continue1 = 1;
    % Condition for keeping same indexed values of i,j, and k

    while continue1 == 1
        delta1 = abs(z3(j) - z3(i));
        % Calculate elevation difference between points i & j (i & i+1)
        delta2 = abs(z3(k) - z3(j));
        % Calculate elevation difference between points j & k (i+1 & i+2)
        if delta1 > 10 & delta2 > 10
            % If the elevation differences (deltas 1 & 2) are greater than 10 feet, then delete point j in elevation
            % and station data
            z3(j) = [];
            s(j) = [];
            continue1 = 1;
            % Do not increase index since a row has been deleted, continue1 is true stay in current "while" loop
        elseif delta1 > 10 & delta2 < 10
            % If the elevation difference delta1 is greater than 10 feet and the elevation difference delta2 is less
            % than 10 then delete point i in the elevation and station data
            z3(i) = [];
            s(i) = [];
            continue1 = 1;
            % Do not increase index since a row has been deleted, continue1 is true stay in current "while"
            % loop
        else
            continue1 = 0;
            % Increase index because no rows were deleted, continue1 is false exit current "while" loop
        end
    end
    n = length(z3);
    % Recalculate the length of the elevation/station arrays
    if k == n
        % Check to see if the end of the arrays have been reached, if they have then continue2 is false and the
        % large "while" loop is exited
        continue2 = 0;
    end
end
end

```

```
%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%
```

```
M = max(s);
% Finds the maximum station in the smoothed data points
```

```
newx = 0:10:M;
newx = newx';
lambda = 2;
x = s;
y = z3;

%function g = loess(s,z3,newx,alpha,lambda,robustFlag)
% curve fit using local regression
% g = loess(x,y,newx,alpha,lambda,robustFlag)
% apply loess curve fit -- nonparametric regression
% x,y data points
% newx,g fitted points
% alpha smoothing typically 0.25 to 1.0
% lambda polynomial order 1 or 2
% if robustFlag is present, use bisquare
% for loess info, refer to Cleveland, Visualizing Data
```

```
% Copyright (c) 1998 by Datatool
% $Revision: 1.00 $
```

```
robust = 0;
if nargin>5,robust=1;end
```

```
n = length(x);
% number of data points
%q = floor(alpha*n);
%q = max(q,1);
%q = min(q,n);
% used for weight function width
q = span1;
tol = 0.003;
% tolerance for robust approach
maxiterations = 100;
```

```
% perform a fit for each desired x point
for ii = 1:length(newx)
    deltax = abs(newx(ii)-x);
    % distances from this new point to data
    deltaxsort = sort(deltax);
    % sorted small to large
    qthdeltax = deltaxsort(q);
    % width of weight function
    % arg = min(deltax/(qthdeltax*max(alpha,1)),1);
    arg = min(deltax/qthdeltax,1);
    tricube = (1-abs(arg).^3).^3;
    % weight function for x distance
    index = tricube>0;
    % select points with nonzero weights
```

[illegible]


```

    MinNPZ = 400;
elseif speed == 30
    MinNPZ = 500;
elseif speed == 35
    MinNPZ = 550;
elseif speed == 40
    MinNPZ = 600;
elseif speed == 45
    MinNPZ = 700;
elseif speed == 50
    MinNPZ = 800;
elseif speed == 55
    MinNPZ = 900;
elseif speed == 60
    MinNPZ = 1000;
elseif speed == 65
    MinNPZ = 1100;
elseif speed == 70
    MinNPZ = 1200;
end

```

```

%% %% %% %% %% %% %% %% %% %% %% %% %% %% %% %% %% %% %% %% %% %% %% %% %% %% %% %% %%
%% %% %% %% %% %% %% %% %% %% %% %% %% %% %% %% %% %% %% %% %% %% %% %% %% %% %% %% %%

```

```

Count1 = 0; % Counter
g = g';
lengthg = length(g);

```

```

for jj = 1:(lengthg - 1 - MinNPZ/10)

```

```

    Count1 = Count1 + 1;
    d = newx(jj);
    % Station of driver location
    dh = g(jj) + 3.5;
    % Elevation of driver eye: "surface elevation" + 3.5 feet

```

```

    if (newx(lengthg)- d) <= MinNPZ
        PZ(Count1) = 1;
        Station(Count1) = M;
        PZ(Count1 + 1) = 0;
        Station(Count1 + 1) = 60606060;
        PZ(Count1 + 2) = 0;
        Station(Count1 + 2) = 60606060;
        break
    end

```

```

    kk = jj;
    continue10 = 1;

```

```

        while continue10 == 1
            % Loop through incremental distances to ensure
            % that sight valleys are detected
            kk = kk + 5;

```

```

os = newx(kk);
% Station of object location
oh = g(kk) + 3.5;
% Elevation of object: "surface elevation" + 3.5 feet

if (os - d) >= MinNPZ
    PZ(Count1) = 1;
    Station(Count1) = d;
    break
end

C = [d, 1; os, 1];
% Slope & intercept coefficients of sightline
E = [dh; oh];
% Elevation coefficients of sightline
D = C^(-1) * E;
% Solve for slope and intercept

increments = d:10:os; % Create array of test stations
increments = increments';
sightline = increments*D(1) + D(2); % Calculate sightline elevations
roadway = g(jj:kk); % Calculate roadway elevations
Test = (sightline > roadway); % Compare sightline and roadway

if min(Test) == 0
    % If min = 0, then there is a sight vertical sight obstruction
    PZ(Count1) = 0;
    Station(Count1) = d;
    break; % 0 indicates a No-Passing Zone, place in PZ array
elseif ((os-d) >= MinNPZ)
    PZ(Count1) = 1;
    % 1 indicates a Passing Zone, place in PZ array
    Station(Count1) = d;
    continue10 = 0;
    break
    % Place the station of the determined passing/no-passing
    % condition above in the Station array
end
end

end

%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%
%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%

Count2 = 0;
Count3 = 1;
continue3 = 1;
continue4 = 1;
n = length(PZ);

while continue3 == 1
    Count2 = Count2 + 1;

```



```

if PZ(Count2) == 0
% If at PZ(Count2) there is a sight obstruction (PZ(Count2) == 0) then complete following if statement
NPZstations(Count3,1) = Station(Count2);
% NPZstations lists beginning and ending stations of No-passing Zones, this line would be a station
% for the beginning of a No-Passing Zone
Count3 = Count3 + 1;
% Set Count3 so that the next station determined will be the end of the No-passing Zones
while continue4 == 1
% Stay in this "while loop" as long as continue4 is true (==1)
Count2 = Count2 + 1;
% Go to next line in PZ to see if there is a sight obstruction
if Count2 == n
% IF PZ = n, the length of the array then the end has been reached and the "while loop" should be
% exited
NPZstations(Count3,1) = Station(Count2);
% This was added, need somehow to flag if this happens
break
end
if PZ(Count2) == 1
% If PZ == 1, then a No-passing zone no longer exists
NPZstations(Count3,1) = Station(Count2-1);
% Go back one station in the Station array and report this as the end of the current No-Passing
% Zone
continue4 = 0;
% The "while loop" should be exited
end
end
Count3 = Count3 + 1;
% Add to Count3 so that next value reported in the NPZstation array will be the start of a new
% No-Passing Zone
end
if Count2 == n
% Check to see if the end of the PZ array has been reached, if so exit the current "While loop"
continue3 = 0;
end
continue4 = 1;
% Reset continue4 to true for the inner "while loop"
end

%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%
%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%

Count1 = 2;
continue1 = 1;
continue2 = 1;
%NPZstations = NPZstations';

while continue1 == 1
% This while loop evaluates the no-passing zones established in the previous checks to see if the distances
% between successive no-passing zones is less than or equal to 400 feet
while continue2 == 1
if (NPZstations(Count1+1,1) - NPZstations(Count1,1)) <= 400

```

[illegible]

Steps for Running No-Passing Zone Program

1. Load MCR Installer on destination machine
2. Load GPS data into GPS_Conversion.xls
3. Eliminate extra data points
4. Convert GPS Latitude/Longitude Data into Northing, Easting, and Station Data
5. Copy station and elevation data into NPZinput.xls
6. Run No-Passing zone program
7. Open NPZoutput.xls to view results and save the file as another name.

* All files except for MCR Installer can be found in the No Passing Zones folder through the following path: **NoPassing_Final\distrib**

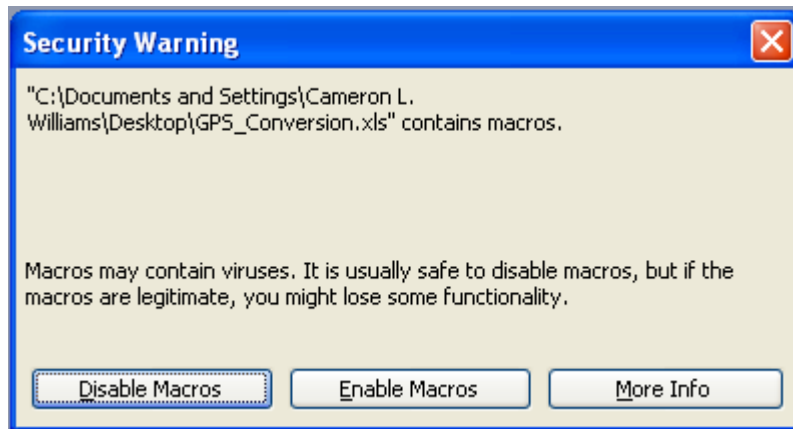
1. Load MCR Installer on destination machine

In order for the compiled MATLAB No-Passing Zone program to work on a machine that does not have MATLAB software installed, MCR Installer must be loaded on the destination machine. This installation process only needs to be done once.

MCR Installer is an executable file and is located on the provided disc. Save the file to the desired computer's desktop. Next double click the file and follow the installation steps. **Important: You must have administrator's right on the computer to install MCR Installer. Otherwise, MCR Installer will not be able to make the necessary changes in order for the compiled program to run.**

2. Load GPS data into Excel Spreadsheet

- Take the data collected from a single GPS run on the roadway and import it into **Raw Data – Step 1** TAB (worksheet) in the **GPS_Conversion.xls** workbook
 - When the **GPS_Conversion.xls** workbook is open popup **Security Warning** will appear. Click **Enable Macros**.



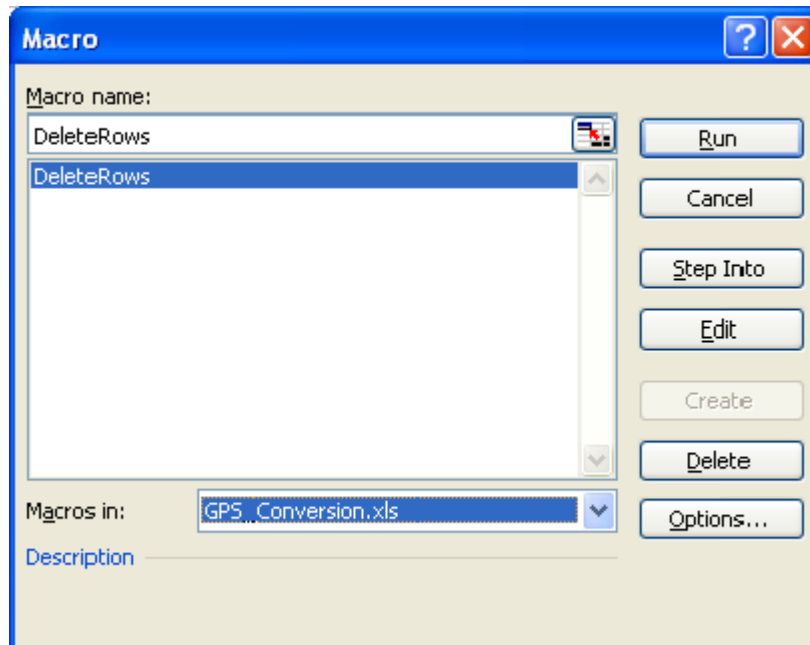
- Next select and copy data from **Raw Data – Step 1 (Tab 1)** worksheet into **Cleaned Data – Step 2 (Tab 2)**. The data copied should include time of collection, longitude in minutes.decimal minutes (mm.mm), latitude in mm.mm, and elevation in m. Longitude values may be labeled x absolute and the latitude values may be labeled y absolute. The first row of copied values should be the beginning of the run. This is only 4 columns of data the data that is copied should only be numeric, no cells with text values should be copied.

3. Eliminate Extra Data Points (Optional)

- If data collected a value other than 1 Hz (1 time per second) follow this step. This will clean the data down to 1 data point every second. Alternatively, the data points collected at speeds greater than 1 Hz (5 Hz, 10 Hz ect) can be used, but the no-passing zone algorithm has been run on data collected every one second.

Note: 1 Hz means the time between data points will be an even one second. If the data was collected at 10 Hz, then there will be 0.1 second between each data point.

Go to the **Tools → Macro → Macros** (or type Alt+F8). Make sure the **Macros in: GPS_Conversion.xls** is selected in the drop down box. Next, select the **DeleteRows** macro and hit **Run**. This macro will clean the data such that the only data points left will be one second apart. If the data was collected at something other than 10 Hz, the macro will need to be modified.



4. Convert GPS Latitude/Longitude Data into Northing, Easting, and Station Data

- Open the worksheet entitled **Stations & Elevations – Step 3 (Tab 3)**. The first four columns should reference the columns in Cleaned Data – Step 2.

First clear the contents of **Row 3** from **Column A to Column J**. Do this for the entire sheet. The first two rows are protected and cannot be changed.

If desired, place the height at which the GPS antenna was on the data collection vehicle. This can be done in **Cell L2** (shaded light green).

Next, select **Cell A2** and copy it down column **A2** until it presents values equal to zero (i.e. it reaches the end of the data on the **Clean Data – Step 2** worksheet).

Select **Cells A2 – J2**. Double click the bottom right corner of the selection to copy the formulas down to the end of the collected data. Raw station and elevation data are shown in **Columns J and K**.

5. Copy Station and Elevation Data into NPZinput.xls

- Now open the NPZinput.xls file. Make sure that **Columns A & B** are clear of data.

Copy the Station & Elevation data (numeric only) from **Columns J & K** in worksheet **Stations & Elevations – Step 3** of the GPS_Conversion.xls file into Columns A & B of the **NPZinput.xls** file.

The design speed/speed limit of the roadway can be indicated in **Cell C2** (shaded light green) in this worksheet. Just click on this cell and select a speed (mph) from the drop down list.

SAVE the **NPZinput.xls** file.

6. Run No-Passing Zone Program

- Run the no-passing zone program by double clicking on the **NoPassing_Final** executable file.

A command prompt box will appear and run. When the command prompt box closes, the program has finished running. Below is an example of the command prompt box.



7. Open NPZoutput.xls to View Results

- Once the no-passing zone program has finished running, open the NPZoutput.xls file to view the results. The sheet named **Start & End** shows the beginning and

ending of each of the identified no-passing zones along with the length of each of the no-passing zones. Dimensions are given in feet and are measured from the beginning of the collected data.

The sheet named **Stations & Elevations** contains the data points of the roadway profile.

The sheet named **Profile** has the graphical view of the roadway profile and should be checked for accuracy.

Use the **SAVE AS** command from the **File** menu to save the file to another name. The data in the **NPZoutput.xls** file can then be deleted and the file used again.

Compiling m-files in MATLAB

The steps below outline the process for compiling an m-file in MATLAB into C-code for distribution to an end user who does not have MATLAB installed on their computer. The end result is an executable file that can be run and which generates the desired output from the m-file. Bold typeface indicate required user actions.

- The MATLAB m-file must be in a function format.

```
function [output] = myfunc ()
% Calculations .....
```

- a. The [output] is not necessary and in order to retrieve output from the compiled file it is recommended to have the m-file print the output in another file such as an Excel file (.xls)

- To begin the compiling process in MATLAB you must first specify the compiler that is to be used. Type the following at the command line and follow the prompts.

```
>> mbuild -setup
```

Please choose your compiler for building standalone MATLAB applications:

Would you like mbuild to locate installed compilers [y]/n? **y (Enter)**

Select a compiler:

[1] Lcc-win32 C 2.4.1 in C:\PROGRA~1\MATLAB\R2007a\sys\lcc

[2] Microsoft Visual C++ 2005 in C:\Program Files\Microsoft Visual

Studio 8

[0] None

Compiler: **1 (Enter)**

Please verify your choices:

Compiler: Lcc-win32 C 2.4.1

Location: C:\PROGRA~1\MATLAB\R2007a\sys\lcc

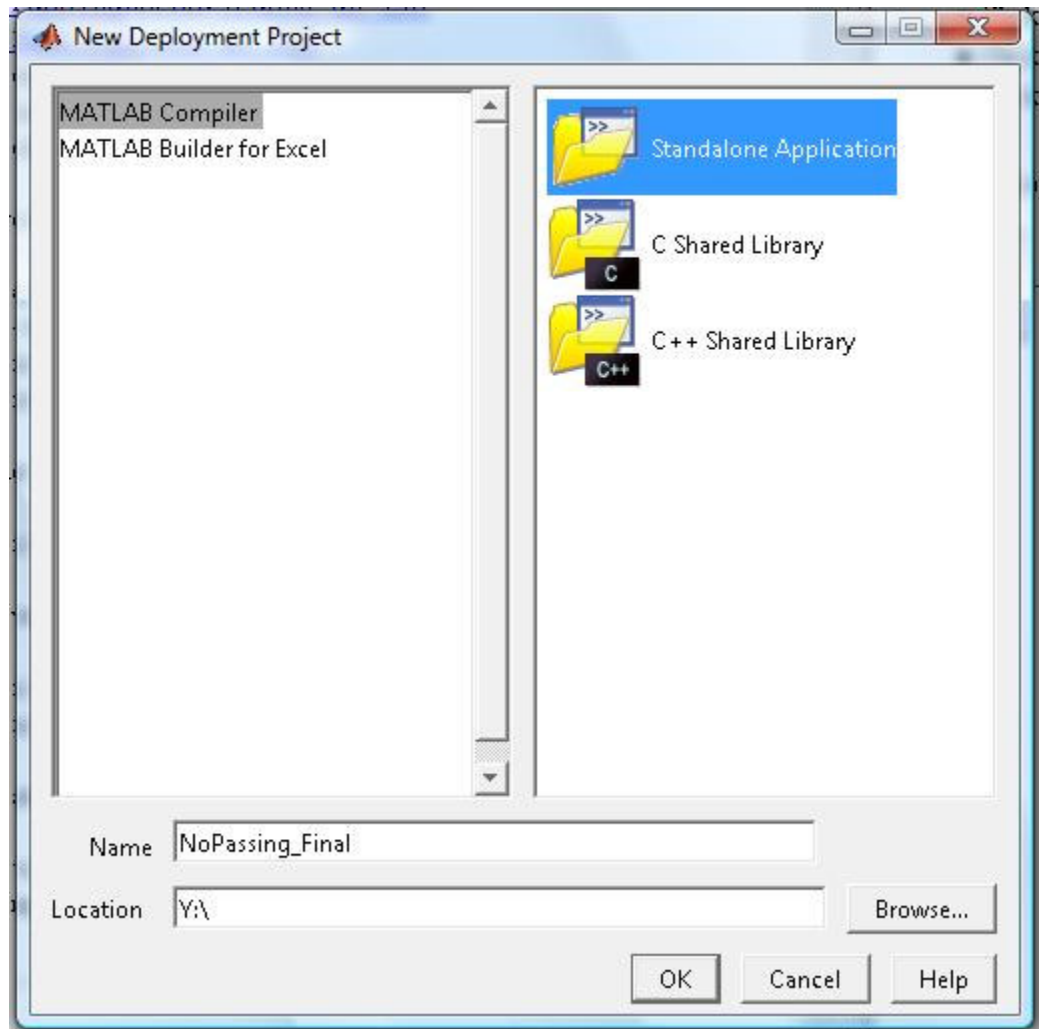
Are these correct?([y]/n): **y (Enter)**

- Next, the compiling process can begin. The compiling GUI will be used. Type the following at the command prompt.



>> deploytool

- Once the GUI is loaded, begin a new project by completing the following:
File → New Deployment Project.

Select: **MATLAB Compiler, Standalone Application**



Create a name for the new project.

- There will be three sub-file folders created.
Main Function, Other Files, C/C++ files
- Drag and drop the m-file to be compiled from the Current Directory into the Deploytool GUI. The file will be placed in the Main Function sub-file folder.
- Next, compile the m-file by building it. Click on the **Build the project** icon. 
- After, building the m-file package the necessary files into a compressed zipped file for distribution. This is done by clicking on the **Package the project** icon. 
- For distribution, the only file that is needed is the compressed package file. Once uncompressed, the compiled m-file is now an executable application. The

packaged project contains the executable application and the MCRInstaller (MATLAB Component Runtime Installer). This should automatically install; however, the user should have administrative privileges on the computer on which MCRInstaller is being installed on. MCRInstaller changes some settings on the computer which allows the compiled m-file to call functions that are typically executed in MATLAB.

NoPassing Application Specifics

- With the packaged file the Excel file filename1.xls should be included. This is where a roadway's profile station and elevation data should be entered. The station data should be in column A and begin in row 1. The elevation data should be in column B and should correspond to the elevation data. The NoPassing application calls this file and then creates an Excel file called NPZstations which contains the beginning and ending stations of no-passing zones as determined by the NoPassing application.

VITA

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